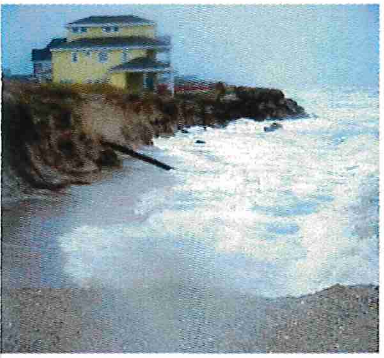
East Ocean View Beach
Nourishment Project



Draft Summary Report
JUNE 2004

City of Norfolk

MOFFATT & NICHOL

EXECUTIVE SUMMARY

In recent years, East Ocean View Beach has suffered severe damage from both long-term erosion and short-term storm-induced erosion. During 2003, the combined impacts of a severe noreaster in April and Hurricane Isabel in September resulted in extreme erosion and threatened properties along the shoreline. Previous beach nourishment projects and/or other hard shoreline stabilization projects implemented by the City have provided some stabilization of the shoreline in discrete locations, but often resulted in updrift and downdrift erosional impacts along the shoreline. Therefore, the City was in need of a full-scale beach restoration project to protect and stabilize the East Ocean View Beach area.

In December 2003, a beach nourishment project was completed along the East Ocean View Beach area, which included the placement of 359,000 cubic yards of dredged material along the shoreline between Little Creek Inlet Jetty and 17th Bay Street (approximately 5300 ft). Since the USACE had a concurrent project for the dredging of Thimble Shoal Channel which allowed for use of material for beach nourishment, this project was completed under the USACE's dredging permit. In addition, a betterment was submitted by VPA on behalf of the City which allowed for deeper channel dredging (to -58 ft MLLW) if needed to meet the project volume. Because of time constraints, the beach fill design was based on the previous template designed by Andrews Miller & Associates for the East Ocean View Study area. The design template included a berm at +4 ft NAVD 88 and a primary dune reaching +8-10 ft NAVD 88 for the study area (final design template used for the construction drawings was modified from the original design based on USACE and contractor concerns and/or requirements). In conjunction with the design and permit application development, a sediment compatibility analysis was completed to investigate the alternative borrow sources in the area, with a focus on Thimble Shoal Channel. Based on the limited sediment data availability, the analysis showed that the Thimble Shoal Channel sediment (in specific areas) would be relatively compatible with the native beach. The final cost to the City of Norfolk for the project was $2,559,274 (USACE letter dated Jan 22, 2004) and all costs associated with the beach nourishment project were 100% non-federal costs.

The East Ocean View beach nourishment project was planned and completed on an accelerated schedule which did not allow for any coastal modeling or analyses prior to design and construction. Therefore, following the beach fill project at East Ocean View Beach, Moffatt & Nichol was tasked with determining the expected design life of the beach fill. This study involved extensive data collection, data transformation through coastal modeling, and short-term and long-term modeling of potential shoreline impacts on the constructed beach.

The overall coastal modeling analysis included the use of the SBEACH model to evaluate the immediate cross-shore loss of sand over a one year time period and following a storm event represented by Hurricane Isabel. Additionally, the GENESIS model was used to evaluate the long-term change in shoreline position based upon a twenty-year time period of wave action. Both modeling analyses required complete nearshore wave data time series broken into sea and swell components, which was developed through the transformation of measured wave data at the Duck FRF offshore of North Carolina. The expected design life was estimated by calculating the time period by which the post-project equilibrium shoreline reached the pre-project, pre-Isabel (June 2003) shoreline position. In addition to the analysis of expected design life of the project, separate SBEACH model results gave insight to the impact Hurricane Isabel would have on the nourished project site.

Through the integrated modeling analyses, it was determined that the design life can be expected to vary across the site with two main critical areas: 1) near Ships Cabin and immediately west of the breakwater field near the end of the project and 2) near the concrete beach structure located at the middle of the project area just seaward of 25th Bay Street. The models estimate that the shorelines in these areas would reach the pre-project position within 7 – 8 years. The SBEACH modeling of Hurricane Isabel yielded an average 35 ft erosion setback at the +3 ft NAVD 88 contour. Therefore, the project design life could be lessened to 4-5 years if storm-induced erosion similar to that estimated with the Hurricane Isabel SBEACH model occurs. While recovery of the beach can be expected to occur following storm-induced erosion, if the storm event occurs during years 4-5, the shoreline could erode to the pre-project position at the critical locations. In any case, some areas along the overall study extent can be expected to have a longer design life than listed above as shown by the long-term modeling and the potential Hurricane Isabel impacts.

As for future recommendations, M&N strongly suggests that the City consider installing wave gages offshore to acquire more accurate wav data which would allow for greater confidence and efficiency in the decision of future shore protection projects. Also, in order to quantify and compare the modeled design life against measured beach changes over time, M&N recommends that the City continue collecting profile surveys every six months. The spacing of the profiles should be such that profiles are collected immediately behind and between the breakwaters (spacing approximately 200 ft).

In summary, the East Ocean View beach nourishment project was implemented successfully, providing crucial shoreline stabilization following the severe storm impacts which threatened the shoreline and structures impending on it. This project can be expected to have a reasonable design life, which will provide for further storm protection and mitigate potential damages posed by future long-term erosion and short-term storm induced erosion. The modeling analyses performed in the comprehensive study of East Ocean View Beach not only provided more insight to the expected design life of the beach nourishment project, but improved the overall understanding of the coastal processes occurring in and around the study area, which will aid in future decision making related to shoreline stabilization and improvement. Furthermore, the data collection and transformation involved in this study will be utilized in subsequent studies of other sections of the City of Norfolk shoreline.

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A. PROJECT BACKGROUND

The City of Norfolk shoreline lies on the Chesapeake Bay and extends approximately 7.3 miles, from Little Creek Inlet to Willoughby Spit (**Figure I-1**). The East Ocean View Beach section of the Norfolk shoreline, previously defined as a critical area of concern for erosional damage (Andrews, Miller & Assoc., Inc, Jan 1993), extends from Little Creek Inlet approximately 5300 feet northwest towards Willoughby Spit. East Ocean View Beach has suffered severe damage from both long-term erosion and short-term storm-induced erosion. In particular, northeast storms (noreasters) have significantly impacted this shoreline in the past, resulting in extreme sand loss and threatened properties along the shoreline. Furthermore, construction of the jetties at Little Creek Inlet is believed to have resulted in a loss of sand along the shoreline west of the jetty, due to interruption of the westerly longshore transport of sand in this area (USACE, 1983). The East Ocean View Beach area is currently the site for a proposed major redevelopment project to include new residential and commercial development along the shoreline. Therefore, this area has become an increasing concern for maintaining adequate protection of existing and proposed structures and improving the quality of the shoreline.

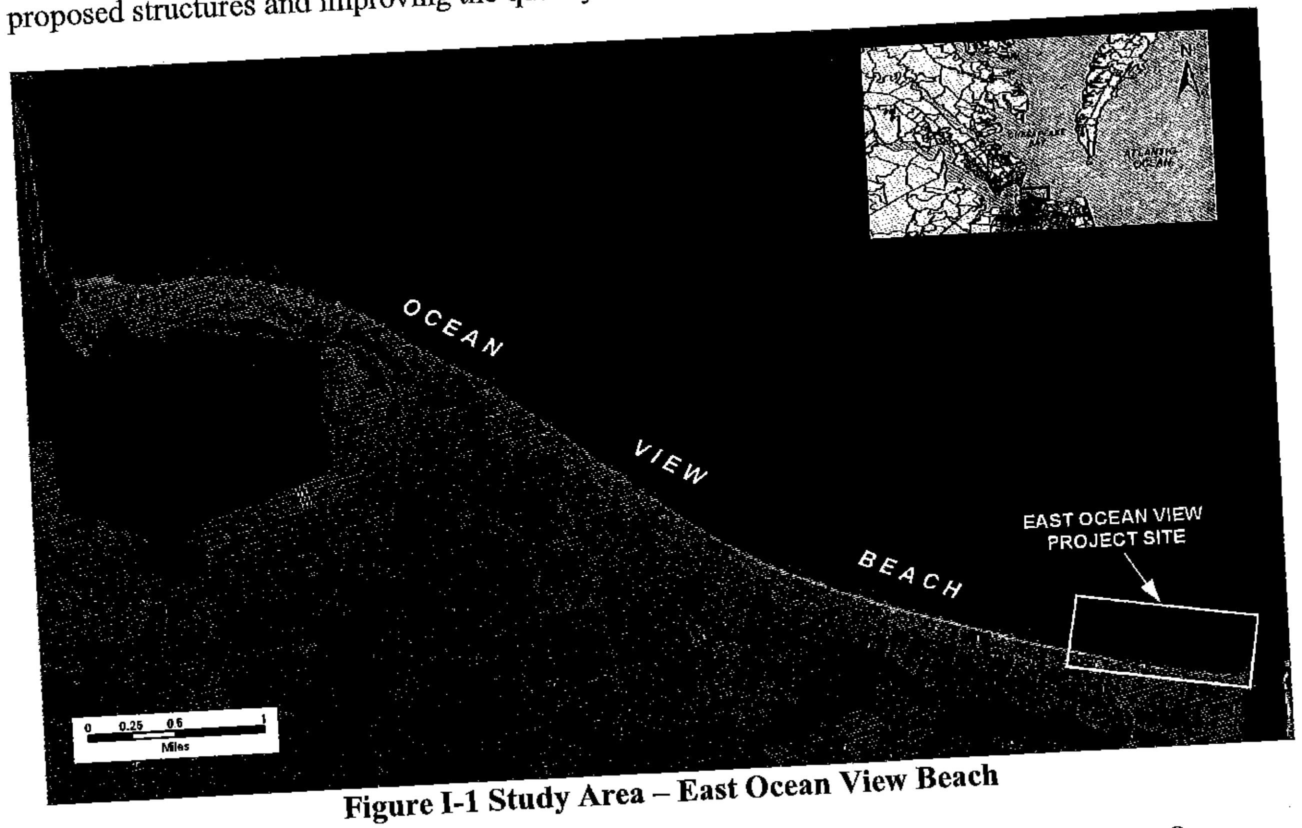


Figure I-1 Study Area – East Ocean View Beach

Previous efforts by the City of Norfolk to reduce storm-induced erosion along the East Ocean View Beach shoreline have included numerous beach nourishment projects (1953, 1960, 1984, 1989, 2003), construction of offshore breakwaters (2000-2001), and various hard shoreline stabilization projects at specific areas of concern (e.g. placement of concrete rubble on beach) (See **Figure I-2**). The previous beach nourishment efforts were typically one-time nourishment projects, occurring either under Federal control or in combination with a local dredging project and were successful at providing short-term reduction of erosion in the area. However, given the projects were not followed up with adequate monitoring or renourishment, they were typically followed by accelerated erosion.

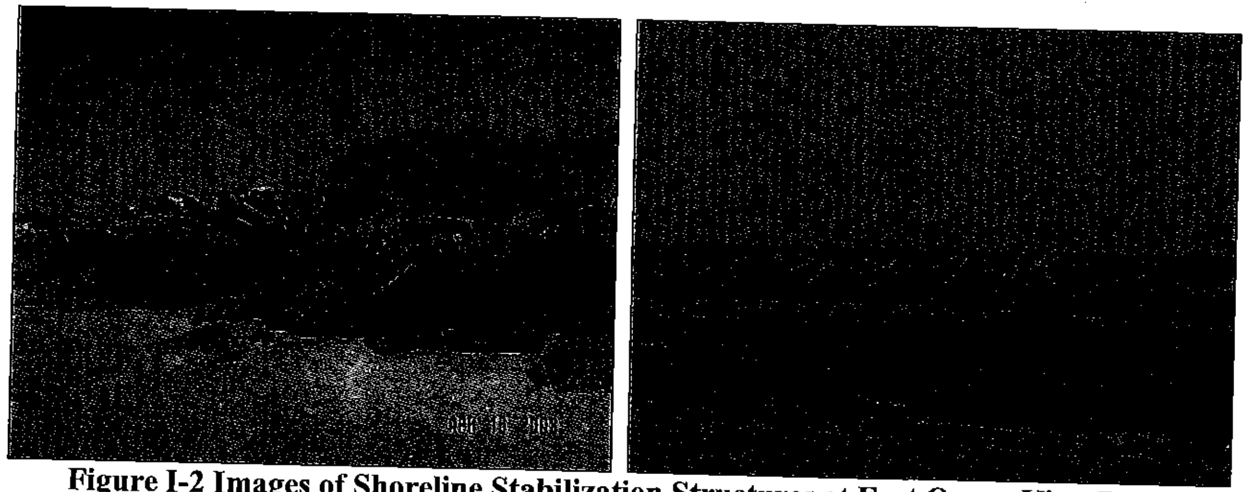


Figure I-2 Images of Shoreline Stabilization Structures at East Ocean View Beach: Concrete Rubble on Beach and Offshore Breakwater (April 2003)

In April 2003, a significant northeast storm hit the City of Norfolk shoreline and resulted in severe erosion along the East Ocean View Beach area and left several existing structures impending on the shoreline. **Figure I-3** shows the conditions of portions of East Ocean View beach following the storm event (photos taken during a site visit in April 2003). In addition, Hurricane Isabel, which hit in early September 2003, caused extensive damage along the entire Norfolk shoreline and surrounding areas.

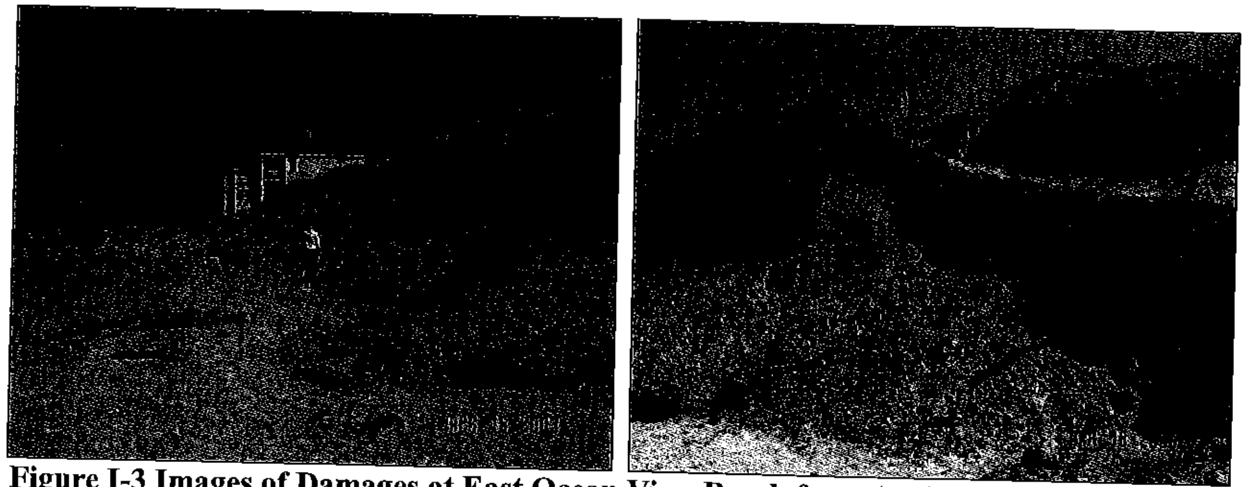


Figure I-3 Images of Damages at East Ocean View Beach from April 2003 Northeast Storm

As a result of these storms, the City of Norfolk declared emergency conditions and that an immediate beach nourishment project in this area was a necessity. At that time, the City became aware that the USACE was planning to dredge Thimble Shoal Channel in the Fall 2003. After consulting with the USACE, it was determined that there was a clause within the permit which allowed the dredged material to be used for beach nourishment. Therefore, the City initiated a beach nourishment project for the East Ocean View Beach coincident with the USACE's dredging project at Thimble Shoal Channel. The project had to be completed under the USACE's dredging permit, as there was not enough time for the City to get permit approval for dredging. However, there were concerns that not enough suitable material was available above the permitted dredge elevation of -52 ft Mean Lower Low Water (MLLW).

In August 2003, the City contacted the Virginia Port Authority (VPA) and asked that they use their Project Cooperation Agreement with the USACE to request a design and construction betterment on behalf of the City. The betterment included dredging the channel to a maximum depth of -58 ft MLLW along a portion of the channel and placing the material along East Ocean View Beach in order to widen it. The VPA also provided up-front funding from the Commonwealth of Virginia's 50-Foot Inbound Element escrow account.

All of this information was then incorporated within revised permit applications for the beach fill. Given the accelerated schedule required to submit the permits and design drawings to incorporate this project with the USACE's dredging project, there was limited time available to complete a modeling analysis and design prior to construction. Therefore, the beach fill design was based on the previous template designed by Andrews Miller & Associates for the East Ocean View Study area. The design template included a berm at +4 ft NAVD 88 and a primary dune reaching +8-10 ft NAVD 88 for the study area. The permit applications were completed in early August 2003. As part of the permit requirements, M&N completed a sediment compatibility analysis to investigate the alternative borrow sources in the area, with a focus on Thimble Shoal Channel. This analysis involved the collection of historical and recent sediment data and borings for potential sand sources near the East Ocean View site. A brief report summarizing the sediment compatibility analysis and the findings for East Ocean View is included in **Appendix A**. The analysis showed that the Thimble Shoal Channel sediment (in specific areas) would be relatively compatible with the native beach. However, it should be noted that the available boring data for Thimble Shoal Channel were sparse and completed approximately 20 years ago.

The final construction plans and documents for the beach nourishment project were completed during mid-late September 2003. **Appendix B** contains a copy of the final construction plans submitted by Moffatt & Nichol (M&N). It should be noted that the final design template used for the construction drawings was modified from the original design based on USACE and contractor concerns and/or requirements. Work on the beach nourishment project began on November 11, 2003 and was completed on December 13, 2003. Approximately 359,000 cubic yards of dredged material was placed along the shoreline between Little Creek Inlet and 17th Bay Street (approximately 5300 feet). The approximate volumetric densities of the project ranged from 10 yd^3/ft to 110 yd^3/ft with an average density of 66 yd^3/ft. **Figure I-4** shows representative aerial (Dec 2003) and ground (May 2004) photos of the study area following nourishment. The final cost to the City of Norfolk was $2,559,274 (USACE letter dated Jan 22, 2004) and all costs associated with the beach nourishment project were 100% non-federal costs..

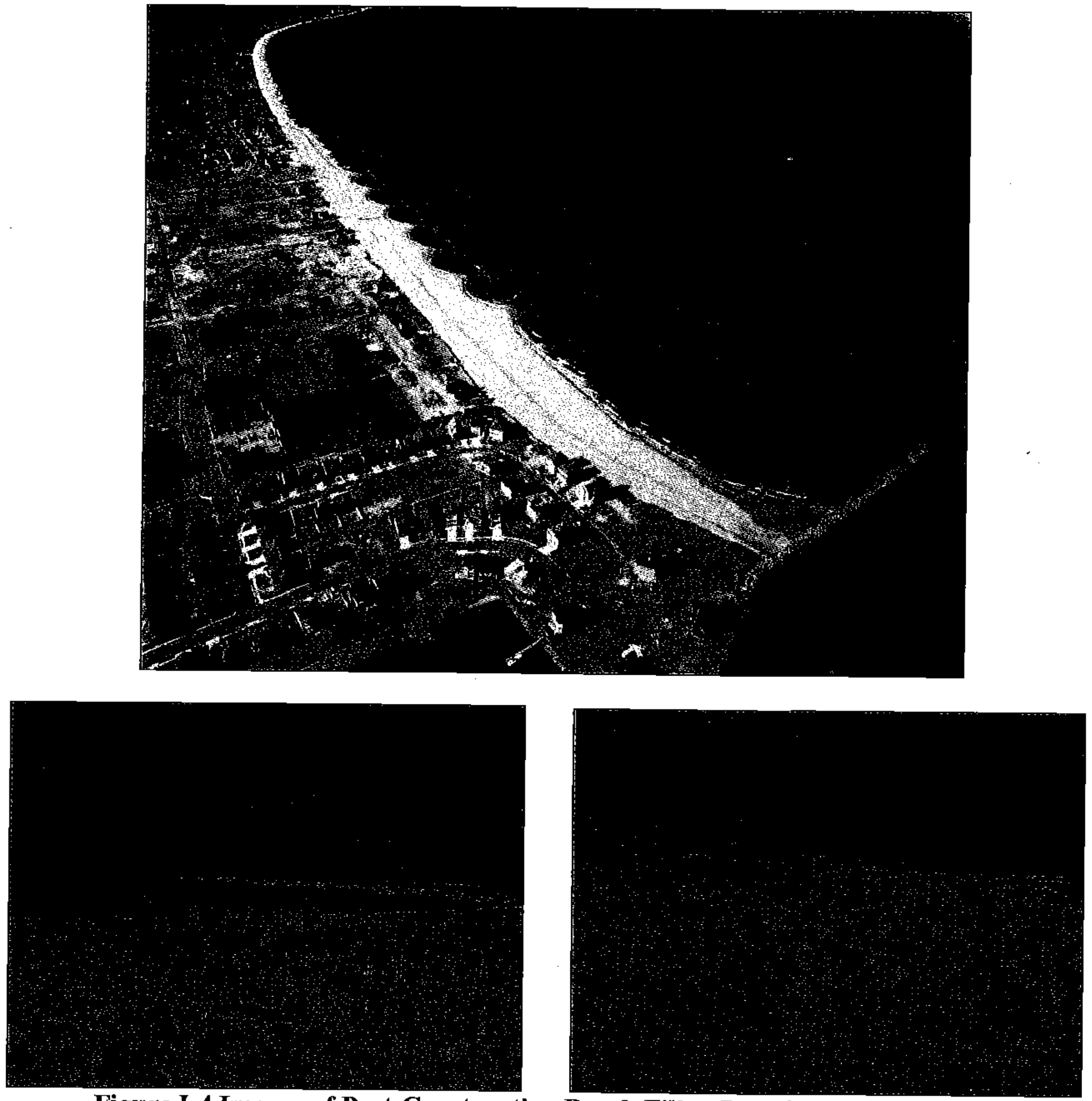


Figure I-4 Images of Post-Construction Beach Fill at East Ocean View Beach (Aerial Photo taken Dec 2003; Ground Photos taken May 2004)

B. SCOPE OF WORK

Following the beach fill project at East Ocean View Beach, Moffatt & Nichol was tasked with determining the expected design life of the beach fill. This task is the subject of this report, including the process of data collection, data transformation through coastal modeling, and short-term and long-term modeling of potential shoreline impacts with the beach fill in place. The analysis of expected design life included the use of two USACE coastal models, 1) GENESIS, a 1-dimensional numerical model which simulates longshore sand transport and movement of the shoreline induced primarily by wave action, and 2) SBEACH, a 2-dimensional numerical model used to simulate cross-shore beach change induced primarily by storm wave action. The scope of modeling completed was as follows:

1. Use SBEACH to evaluate the immediate cross-shore loss of sand in the berm and/or dune for a one(1)-year time period (representative time period from November 2000 through October 2001 was selected based on available wave data) simulation to determine the equilibrium profile. The profiles used in this SBEACH modeling were the post-construction profiles. Using the model results of the 1-year simulation, calculate the equilibrium profile shoreline position to use for the long-term GENESIS modeling to determine project design life.

2. Use GENESIS to evaluate the long-term change in shoreline position based upon a long-term, 20 year period (2003-2023) of wave action. The input wave data used in the long-term model was a representative nearshore wave data set from 1991-2003 (repeated to complete a 20-yr simulation). As stated above, the starting shoreline position was not the immediate post-construction shoreline but the equilibrium profile shoreline position. The GENESIS model included the existing breakwaters and the jetty at Little Creek Inlet and utilized parameters determined during the model calibration process.

3. Using the GENESIS model results, estimate project design life by calculating when the post-project shoreline reaches the pre-project (June 2003, pre-Isabel) shoreline position.

This sequence of modeling predicted how the shoreline can be expected to move and what extent of berm and/or dune loss can be expected to occur immediately following the beach nourishment project and in the long-term. In addition to the analysis of expected design life of the project, separate model results were performed to estimate the impact Hurricane Isabel would have on the nourished project site.

The modeling analysis not only provided more insight to the coastal processes occurring in and around the study area, but will aid in future decision making related to shoreline stabilization and improvement. Furthermore, the data collection and transformation involved in this study can be utilized in subsequent studies of other sections of the City of Norfolk shoreline.

II. DATA COLLECTION

The East Ocean View Beach shoreline was modeled to determine the expected design life, considering both the immediate cross-shore loss of sand and adjustment to an equilibrium profile and the overall long-term shoreline erosion. In order to develop such models, an intensive data collection of coastal conditions, including tide and wave data nearshore and offshore of the study area was necessary. Additionally, beach and bathymetric survey data were collected for developing the cross-shore model input profiles. Shoreline position data obtained from aerial photos and historical maps were also used for developing long-term shoreline evolution model inputs and to calibrate these models. Finally, recent sediment data characterizing the pre-nourishment and current conditions (post-nourishment) of the sediment at the study area were also acquired.

The defined coordinate system utilized, where applicable, in this study was State Plane, Virginia South (Zone 4502), with units in feet. The vertical datum used for the survey data was NAVD 88. An updated mean high water (MHW) elevation of +0.66 ft NAVD 88 and mean low water (MLW) elevation of -1.94 ft NAVD 88 were assumed for all coastal modeling applications, based on the 1960-1978 tidal epoch benchmarks at the NOAA Chesapeake Bay Bridge Tunnel gage and the Little Creek Inlet gage. Finally, in extracting shoreline elevations an approximate mean tide level (MTL) of -1.0 ft NAVD 88 was assumed.

A. COASTAL DATA

A thorough search was completed to identify all data sources near and around the study area with available measured or hindcast wave data and/or measured tide data. **Figure II-1** is a summary map showing the location of all identified data sources. **Table II-1** summarizes these collected data sets including the type of data available, the time period covered by the data set, and a description of the data set.

As shown on **Figure II-1**, a number of wave data sets were collected and analyzed as part of this study with the goal of developing a finalized wave data set representative of nearshore conditions at the site. The wave data analyses were a major portion of this study and included comparisons of available wave data, development of a final offshore wave data set, and transformation of this offshore wave data to nearshore site conditions. The details of these analyses will be discussed in **Section III** of this report.

The available tide data included a number of gages operated by NOAA National Ocean Service (NOS) within the Chesapeake Bay, and near the study area. The only tide data station shown on **Figure II-1** and on **Table II-1**, is the Chesapeake Bay Bridge Tunnel Gage. Several other gages exist or have existed in the vicinity of the study area, including Little Creek gage and Lynnhaven Fishing Pier gage, however these data sets are only available for limited time periods. Given that the Chesapeake Bay Bridge Tunnel gage contained the most comprehensive data (1975-present), this tide gage was selected for developing the water level input for all modeling aspects of this study. The water level data was converted to a common datum of NAVD 88 using the tidal benchmark data (1960-1978 tidal epoch) available at the Chesapeake Tunnel gage and the Little Creek Inlet gage.

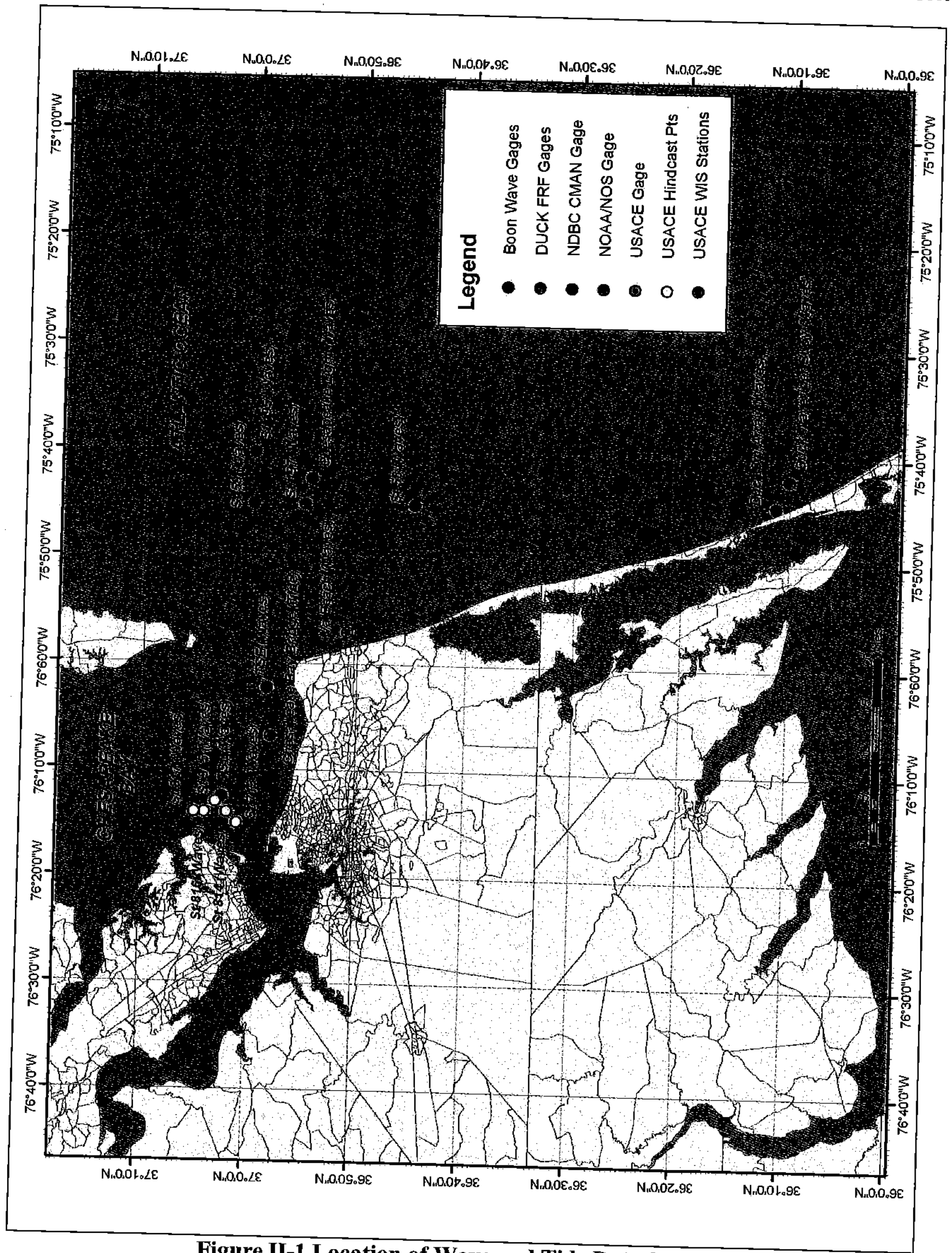


Figure II-1 Location of Wave and Tide Data Sources

Table II-1 Available Wave and Tide Data

SOURCE	STATION(S)	TYPE	TIME PERIOD	DESCRIPTION
Dr. John Boon, Virginia Institute of Marine Science (VIMS)	TSL	wave	Sept 1988-Jan 1995 (intermittent 3-9 month time periods)	***Measured*** 3-hour wave data (burst averaged significant wave heights, peak periods, primary wave directions)
	TSE	wave	4 storm events (2/11/1993,2/26/1993, 3/13/1993, 9/22/1994)	***Measured*** wave data in raw format for 4 storm events
USACE (OCTI)	713, 814, 816, 817, 915	wave	1986 -1995	***Hindcast*** hourly wave data (significant wave heights, peak periods, mean wave directions); reports combined wave parameters and sea and swell wave parameters
USACE CHL	WIS 58, 59	wave	1956-1995	***Hindcast*** 3-hour wave data (significant wave heights, peak periods, mean wave directions); reports combined wave parameters and primary and secondary wave component parameters
USACE CHL	WIS 195, 197	wave	1990-1999	***Hindcast*** hourly wave data (significant wave heights, peak periods, mean wave directions); also reports wave data in raw spectral format
NOAA NDBC	CMAN CHLV2 (Chesapeake Light)	wave	1984-2003 with breaks	***Measured*** hourly wave data (significant wave heights, peak periods)
USACE	VA001	wave	1992-1997, 1999-2001 with breaks	***Measured*** 4-hour wave data (significant wave heights, peak periods)
USACE DUCK FRF	8-meter array: 191, 161, 251, 211, 111	wave	1991 - present	***Measured*** 3-hour wave data (significant wave heights, peak periods, mean wave directions); also measured 6-hr directional spectral wave data available
USACE DUCK FRF	Waverider Buoy 630	wave	Jan 1980 - present	***Measured*** hourly wave data (significant wave heights, peak periods, mean wave directions)
NOAA/NOS Co-ops	8638863 (Chesapeake Bay Bridge Tunnel)	tide	1975-2003	***Measured*** hourly water level data from 1975-2003 ***Measured*** 6-minute water level data from 1992-2003

B. BEACH & BATHYMETRIC SURVEY DATA

Beach and bathymetric survey data obtained by the City of Norfolk and Waterway Surveys & Engineering were used for developing model inputs for both the SBEACH and GENESIS models. The dates and sources of all survey data sets obtained are shown in **Table II-2**. **Appendix C** includes a series of maps showing the coverage and extent of these surveys. Please note that the October 2003 (post-Isabel) surveys were not provided to M&N in AutoCAD/GIS format. Therefore, a coverage map for this survey date is excluded from **Appendix C**.

Table II-2 Beach & Bathymetric Survey Data Summary

DATE	SOURCE
Fall 1998	City Surveys
Oct 1999	City Surveys
July 2000	City Surveys
Oct 2000	City Surveys
Fall 2001	City Surveys
Spring 2002	City Surveys
Fall 2002	City Surveys
March 2003	City Surveys
April 2003	City Surveys
June 2003 (Pre-Isabel)	Waterway Surveys & Engineering
Oct 2003 (Post-Isabel)	Waterway Surveys & Engineering
Nov-Dec 2003 (Post-Fill)	Waterway Surveys & Engineering

The survey data were used for a number of applications in this study including general creation and comparison of beach profiles and calculation of shoreline positions. The availability of numerous survey dates allowed for calibration of the SBEACH model using initial and final profiles that coincided with available wave data. Additionally, the pre- and post-Hurricane Isabel survey data allowed for the development of a calibration model simulating this storm event. Finally, the post-fill surveys, obtained in November-December 2003, served as the model input for the SBEACH model of current conditions. The use of these survey data sets in specific modeling applications will be discussed further in this report.

C. SHORELINE DATA

In addition to the beach and bathymetric survey data, digitized shorelines were obtained for a number of historical and recent dates. A majority of the shoreline data was obtained from a study completed by Dr. David Basco of Beach Consultants, Inc, as part of a comprehensive shoreline analysis for Ocean View Beach completed in January 2004. This study involved the collection and analysis of historical and recent shoreline positions dating 1852 to 2000 and was performed in two phases, beginning with the critical East Ocean View section, and following with the remainder of the 7.3 mile shoreline. The process involved determining the shoreline positions from historical NOAA NOS "T-sheets" (topographic maps) and aerial photographs. **Table II-3** lists the dates, sources and coverage of the data sources for which shoreline position data was collected in the East Ocean View Beach area. The findings of this study are summarized in Dr. Basco's report entitled "Chesapeake Bay Shoreline Study: City of Norfolk, Virginia", dated January, 2004. A full copy of the report can be found in **Appendix D**. The following discussion only pertains to the shoreline data retrieved for the East Ocean View project area.

Table II-3. Data Sources used for Shoreline Study – East Ocean View Beach

DATE	SOURCE	COVERAGE
1852	NOAA T-Sheet T-507 (1:20,000)	Full
1876	NOAA T-Sheet T-1462a (1:20,000)	Full
1884	NOAA T-Sheet T-1659 (1:20,000)	Full
1916	NOAA T-Sheet T-3647 (1:20,000)	Full
1929	NOAA T-Sheet T-4456 (1:10,000)	Full
1937	VIMS aerial photography archive	Partial
1942	NOAA T-Sheet T-8301,02 (1:20,000)	Full
1956	VIMS aerial photography archive	Partial
1963	NOAA T-Sheet T-11704 (1:20,000)	Full
1970	VIMS aerial photography archive	Full
1976	VIMS aerial photography archive	Full
1980	VIMS aerial photography archive	Full
1995	VIMS aerial photography archive	Full
1999	VIMS aerial photography archive	Full
2002	VIMS aerial photography archive	Full

For each of the above data sets, a shoreline was digitized and placed in an AutoCAD format. A baseline for measuring shoreline positions was digitized approximately along Ocean View Avenue at East Ocean View Beach. Transects running perpendicular to the baseline were placed every 180 feet across the shoreline. **Figure II-2** shows the defined baseline and transects used in Dr. Basco's study overlain on March 1999 aerial photography (provided by the City). The distance from the baseline to the shoreline was measured along each transect. Shoreline change rates were computed for the above data sets by dividing the change in shoreline position by the time between each data set.

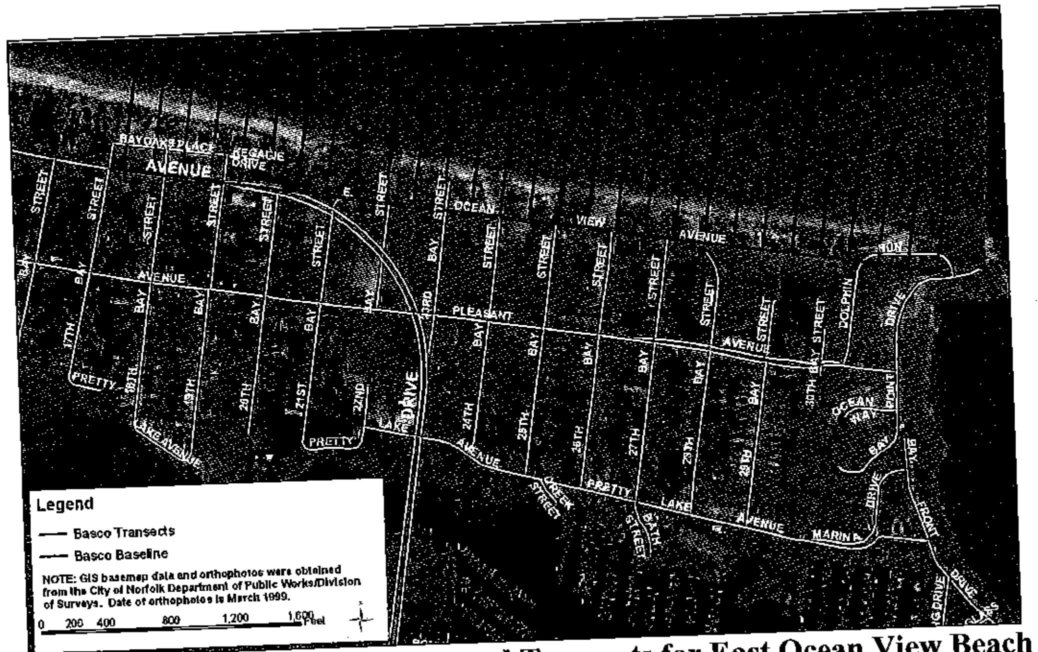


Figure II-2 Dr. Basco's Baseline and Transects for East Ocean View Beach

For the dates ranging from 1852-1980, the shorelines were delineated from the digital T-sheets or the digital aerial photographs. In the event that a digital aerial photography was not georectified, the photo scale and placement was determined using horizontal distances between recognizable features.

The VIMS aerial photography dating 1995, 1999, and 2002 were not in digital format. The procedure used by Dr. Basco to delineate the shoreline positions for these dates involved locating the baseline on the hard copy maps and using recognizable features (e.g. streets) which could be located and scaled from an existing AutoCAD layer, to determine the photo scale. The shoreline offsets (distance from the baseline) were then scaled and adjusted to true scale. The final shoreline was delineated by connecting the line between the measured positions. This resulted in a fairly coarse shoreline because the distance between the actual measured positions (recognizable features such as streets) was significant, approximately 500 ft on average (not coincident with transect positions). For the purposes of the modeling efforts involved in this study, these shoreline delineations were not adequate. Specifically, the resulting shoreline did not capture features that would influence sediment transport, such as the rubble mound seawalls placed along the study area. To alleviate this problem, the hard-copy aerial photographs (1995, 1999, and 2002) were scanned and georeferenced by M&N using ArcGIS software tools and other available georeferenced images for the study area. This allowed for digitizing a more detailed shoreline which captured the existing structures and natural meandering of the shoreline. A comparison of the shorelines developed initially as part of Dr. Basco's study and after georeferencing the image is shown for the 1999 photography in **Figure II-3**.

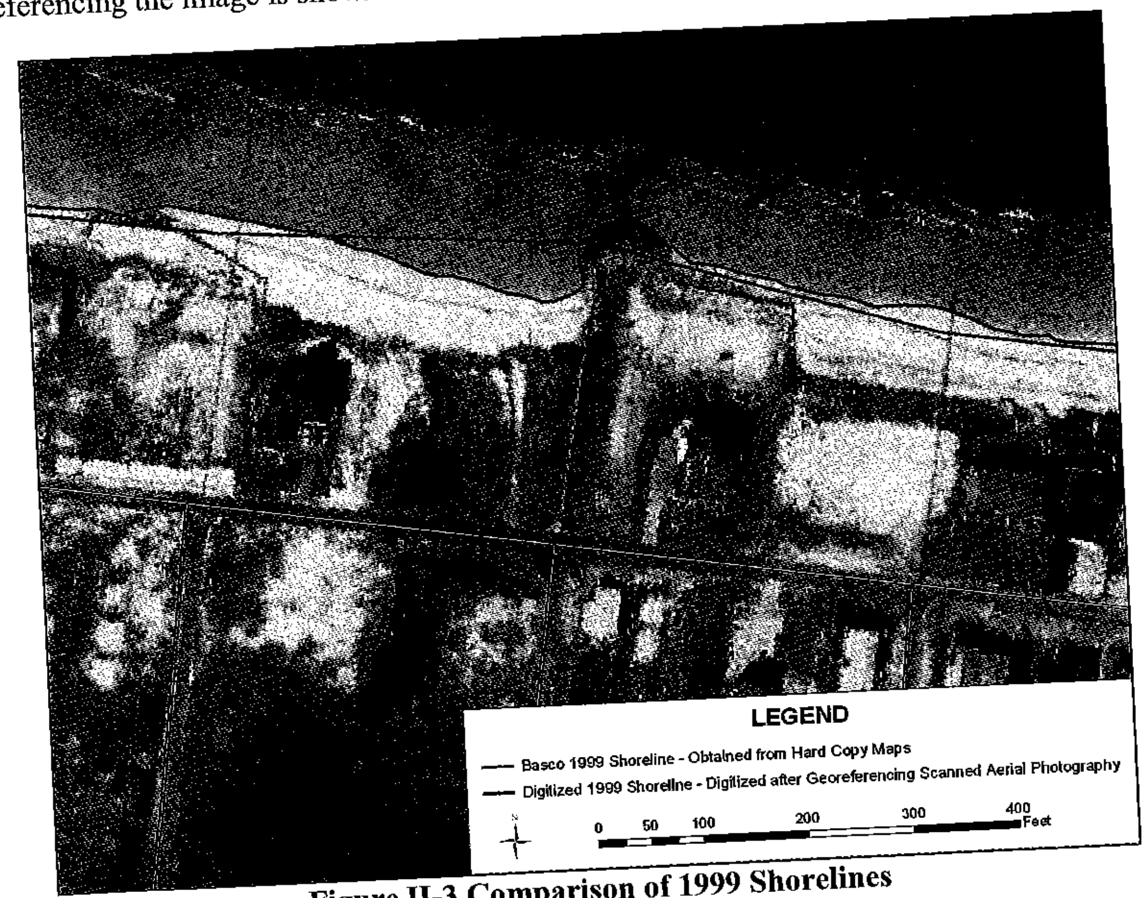


Figure II-3 Comparison of 1999 Shorelines

The digitized shoreline data was used, where applicable, in the GENESIS long-term shoreline evolution modeling. The historical data sets ranging from 1995-2002, served as a basis for calibrating the GENESIS model. The use of shoreline data in specific applications of this study will be discussed in more detail in **Section V** of this report.

D. SEDIMENT DATA

Sediment data were collected at the study area prior to and following the nourishment project by Waterway Survey & Engineering.

The pre-nourishment sediment data was used in determining sediment compatibility between the source sand and the native beach as well as in the calibration models. The sediment data samples taken in June 2003 were spaced approximately every 500 ft along the study area and were taken at the dune crest, mid dune, dune toe, mid beach, -6 ft NAVD 88 water depth, and -15 ft NAVD 88 water depth. These samples were analyzed and summarized to develop a characteristic mean effective grain size for the study area (pre-nourishment, pre-Isabel). This analysis is discussed in more detail in the Sediment Compatibility Analysis Report in **Appendix A**. In summary, an average pre-project representative effective grain size of 0.23 mm was determined for the study area.

Following the beach fill construction, additional sediment samples were obtained every 1000 ft along the study area. This data was summarized in a report dated April 29, 2004. Again, the sediment samples were obtained at mid dune, mid berm, and approximately between the mid-high and mid-low water lines. **Appendix E** contains the sediment grain size distributions for the post-nourishment East Ocean View study area. After summarizing these data, it was determined that the sediment grain size characteristics varied between the eastern and western portions of the study area. Therefore, characteristic sediment grain sizes of 0.5 mm for the eastern section of the study area and 0.34 mm for the western portion of the study area were selected for the models based on the current, post-fill beach conditions

Since the post-fill sediment data were not received until late April, 2004, the completed SBEACH modeling incorporated the pre-nourishment, pre-Isabel effective grain size of 0.23 mm. This will be discussed further in **Section IV**. Since the GENESIS long-term modeling was still in progress at the time the new data were received, the post-fill sediment characteristics were reflected in this modeling task.

III. WAVE DATA DEVELOPMENT & TRANSFORMATION

As stated previously, a number of wave data sources were identified and collected for locations both nearshore and offshore of the site at East Ocean View. The wave data would be used in both SBEACH and GENESIS modeling to estimate the project design life. Therefore, the goal was to develop a comprehensive wave data set that could be used to meet the goals of the different modeling aspects of this study.

Given the unique location of the study area within the Chesapeake Bay and M&N's experience with similar projects in other areas, it was desirable to obtain or develop wave data which was broken into sea and swell wave components. The waves in this area are known to have a consistent bimodal distribution (as determined by Dr. Boon of VIMS), by which the site is impacted by sea waves generated locally in the Bay and swell waves arriving from the Atlantic Ocean. The sea waves are more dominant in the winter months, and are typically shorter-period, steep waves arriving from the North. In contrast, swell waves typically have longer periods (6-18 seconds) and arrive mainly from the east to northeast quadrant. Sea and swell wave components are not often measured or reported, but can be computed from directional spectral wave data which measures wave energy by period and direction.

In addition to developing sea and swell wave components, the modeling tasks necessitated that the wave data be representative of nearshore conditions. Therefore, any wave data time series had to be refracted to a position just offshore of the site. For the purposes of this study, the required nearshore wave condition was defined as -20 ft NAVD 88. **Figure III-1** shows the nearshore wave data input location as defined for the wave data transformation and as required by the coastal modeling applications included in this study.

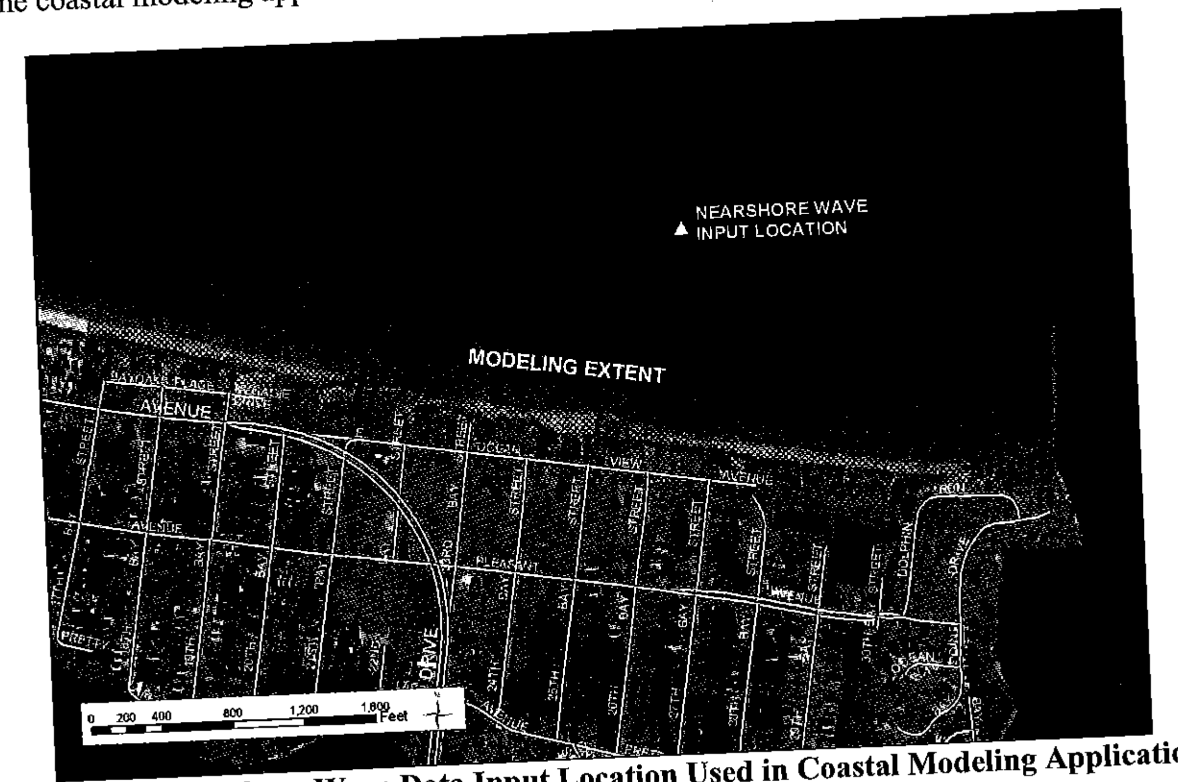


Figure III-1 Nearshore Wave Data Input Location Used in Coastal Modeling Applications

In general, it was preferable to use measured wave data for the modeling tasks. However, hindcast wave data was acceptable if it overlapped and matched well with comparable measured data.

Having defined the overall wave data requirements, the applicability of a particular wave data set to this study was dependent on the specific modeling requirements. Based on the modeling tasks outlined in **Section I-B**, the wave data inputs required for the study included:

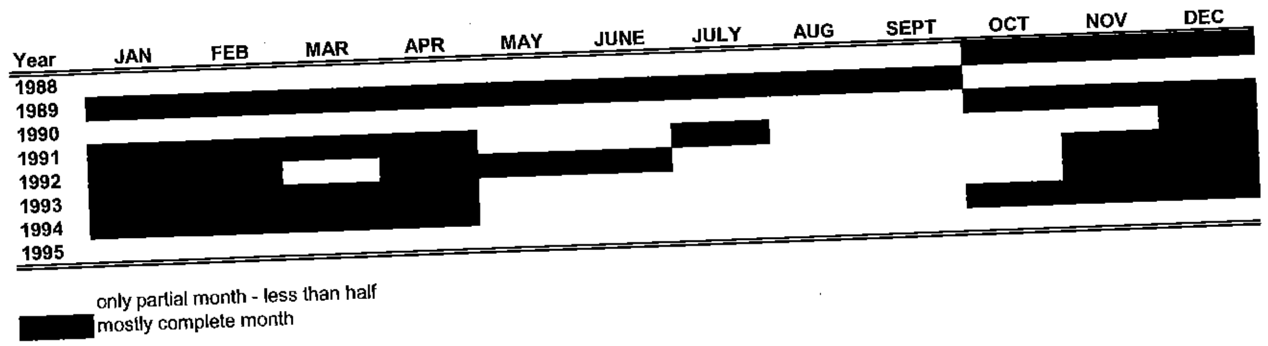
- SBEACH Cross-Shore Storm-Induced Erosion Modeling
  - To estimate initial losses and determine equilibrium profile, representative 1-year wave time series,
  - For calibration purposes, various 6 month to 1-year wave time series coinciding with available measured shoreline profiles (City or Waterway surveys), and
  - To estimate major storm losses following initial 1-year losses, Hurricane Isabel wave time series data.

- GENESIS Long-Term Shoreline Evolution Modeling
  - Long-term (at least 10 years), continuous, reasonably long wave time series, and
  - For calibration purposes, continuous wave data time series coinciding with available measured shoreline positions.

A. ANALYSIS OF NEARBY WAVE DATA SOURCES

Initially, the measured and hindcast wave data closest to the site (**Figure II-1**), including Dr. John Boon's measured wave data at the Thimble Shoal Light (TSL) gage and the USACE wave hindcast data stations (St 713, 814, 816, 817, 915), were considered. The USACE hindcast data was a reasonably long (1986-1995) continuous time series which contained sea and swell components of wave height, period, and direction. The TSL gage data consisted of periodic measurements (typically Sept-April) over a six year time period, which was deemed potentially useful in checking the accuracy of other data sources. **Table III-1** shows the availability of measured wave data from the TSL gage.

Table III-1 Wave Data Availability from Dr. Boon's TSL Gage



Since the hindcast data stations were in close proximity to the measured data at TSL, a comparison of the hindcast and measured data was done to determine the accuracy of the hindcast data. Since Station 915 was closest to the TSL gage, the significant wave heights were compared for these stations during a number of overlapping time periods. In addition, percent exceedance curves of significant wave height were computed for both stations during the coincident time periods. **Figures III-2 and III-3** show a direct comparison of the significant wave height and a comparison of the percent exceedance of significant wave height, respectively, for the USACE and VIMS gage during October 1992 to April 1993.

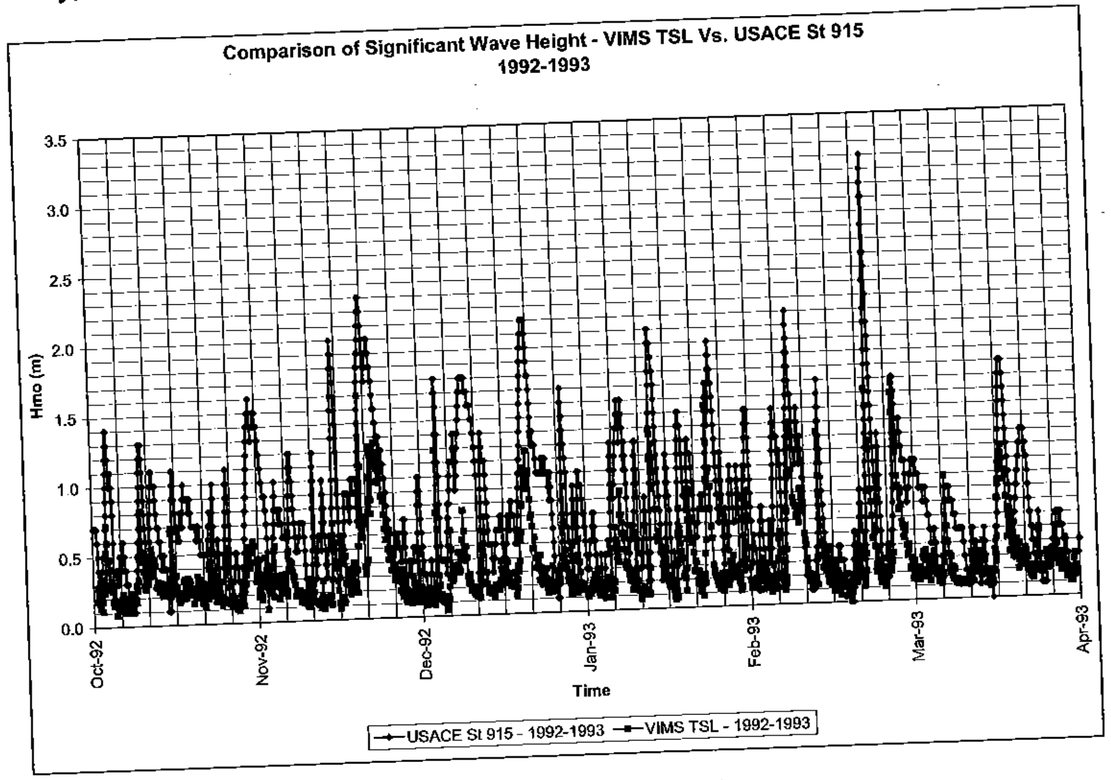


Figure III-2 Comparison of Significant Wave Height – VIMS TSL Vs USACE St 915 – 1992-1993

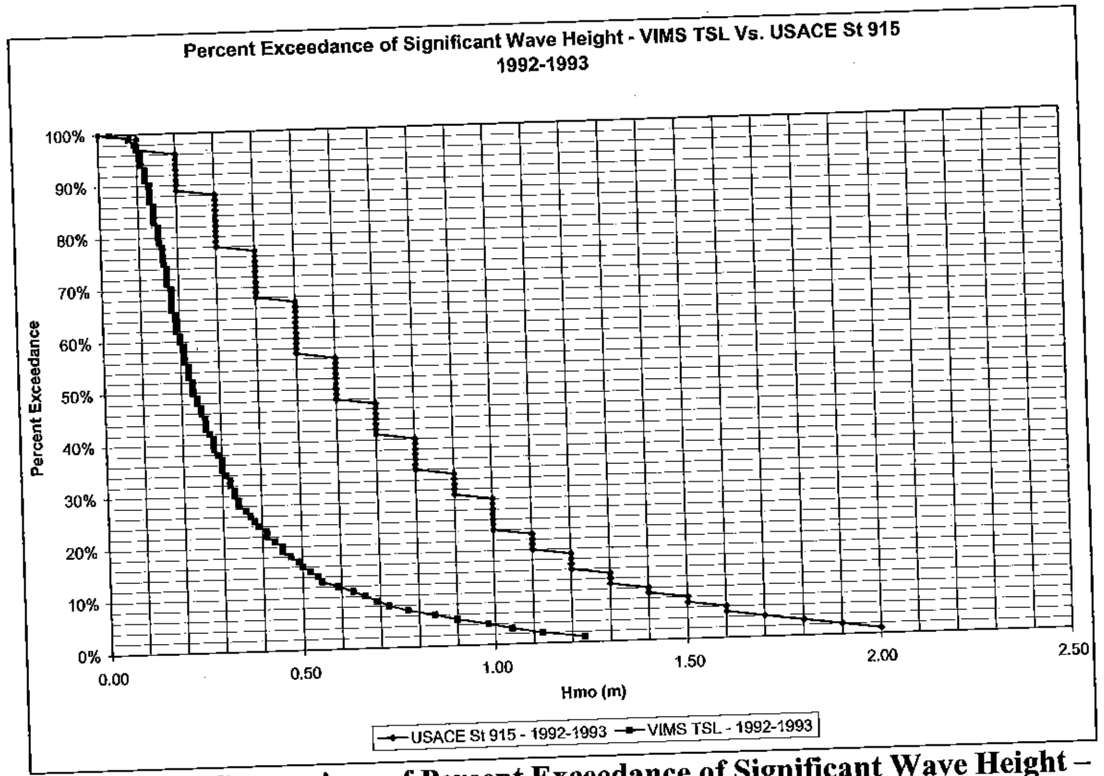


Figure III-3 Comparison of Percent Exceedance of Significant Wave Height – VIMS TSL Vs. USACE St 915 – 1992-1993

As shown, the USACE St 915 wave heights are consistently higher than the measured TSL wave heights. The significant difference between these two data sets brought to question the reliability of either for use in this study. After further investigation of both data sets and discussions with the contractor that performed the hindcast, it was decided that neither was applicable to the study area or for our modeling application based on the following observations:

<u>USACE Hindcast Wave Data</u>

1. USACE Hindcast model (OCTI) was not calibrated for locations near the stations of interest in this study, and
2. USACE swell data contained peak wave period values of zero for a significant number of records

<u>Boon's TSL Wave Data</u>

1. Not a continuous long time series; on average includes Sept – April for each year from 1988 – 1995,
2. No sea and swell wave component data available and no raw wave spectral data available, and
3. Based on percent exceedance analysis, wave heights may be underestimated. Percent exceedance of a 1 meter wave ranged from 1% to 4% for individual time periods analyzed.

In addition to the reasons outlined above, it should also be noted that some preliminary refraction modeling was completed and showed that waves near the project site would likely be somewhat different from the wave climate observed at the TSL and USACE stations. Therefore, offshore wave data sources were then reviewed.

B. ANALYSIS OF OFFSHORE WAVE DATA SOURCES

As an alternative to the nearby measured and hindcast wave data, the use of wave data sources located at stations offshore in the Atlantic Ocean (**Figure II-1**) was explored. The importance of obtaining wave data broken into sea and swell components was imperative in consideration of the offshore wave data because each wave train needed to be uniquely transformed into the Bay and to the site. In fact, M&N's experience with similar projects has shown that unless the wave energy is broken into sea and swell components, there is a distinct probability that important energy directionality and frequency can be missed by the averaging procedures that summary datasets often include. To transform the data sets, the sea wave data (typically short period waves from the North) were factored down by a ratio of fetch lengths between the offshore site and the study area site. Similarly, the swell waves were refracted to the site from the offshore position.

As described, the use of measured wave data was generally more desirable than hindcast data for the modeling applications in this study. The closest offshore measured data to the site, the NOAA CMAN buoy at Chesapeake Light, included data from 1984-2003, but was not broken into sea and swell wave components. As shown in **Table II-1**, the USACE Virginia Beach gage data was quite limited, containing only fractions of data for a given year. Finally, the USACE Duck FRF 8 meter array gage included spectral wave data from 1991 to present, while the waverider buoy only reported the combined wave data. As a potential alternative to measured data, the WIS wave hindcast data at Stations 58 and 59 both contained wave data which was broken into primary and secondary components for the time period of 1956-1995. The primary and secondary components were not synonymous with sea and swell, but allowed for an interpretation of sea and swell with some judgement applied on the separation of the two wave trains. The more recent WIS data at Stations 195 and 197 did not report the sea and swell wave components, however, the raw wave spectral data was available for computing sea and swell. However, the WIS hindcast wave data were based on 3-hour snapshots of the wind field over a 2.5 degree grid. This coarse model could easily miss peak wind speeds and locations which would create the highest wave conditions. By comparison, the Duck FRF data was true measured wave conditions reported every 6 hours.

After reviewing all of the available offshore data sources, the most applicable measured data was the Duck FRF 8-m array directional spectral data. The advantages of using the Duck FRF data include that this measured data consisted of directional spectral measurements and the lengthy time period of measurements met the requirements of several modeling tasks involved in this study. Specifically, the data extended through 2003, providing time series for use in calibration of the GENESIS model which coincided with pre-nourishment conditions on the beach (i.e. allowed for calibrating a model with breakwaters in place). Even though this gage was farthest from the site, the analyses and comparisons with other available data will show that it is the most appropriate for use in this study. This data set was selected with the anticipation that some transformation of the sea and swell components would be performed to bring the waves to a nearshore (-20 ft NAVD 88) location at the site. To validate the use of the Duck FRF wave data, several comparisons were performed with other valid measured and hindcast wave data sets.

C. VERIFICATION OF DUCK WAVE DATA

To complete the verification process, the Duck 8-m array data was first divided into sea and swell components. Spectral wave data can be divided into sea and swell components by selecting a wave period which characterizes the division of the two wave trains. Once a wave period division is established, the spectral wave energy is divided into sea and swell and summed for each time step to develop the significant wave height for each component. As stated previously, the waves in the Chesapeake Bay have a bimodal distribution, with sea waves having shorter wave periods (typically less than 6 seconds) and swell waves having longer wave periods (typically 6-18 seconds). The spectral wave data for the Duck FRF station was summarized by month and viewed graphically in the format of a polar contour plot, which shows the wave energy versus period and direction. **Figure III-4** shows an example of the January plot for the Duck FRF 8-m array data set. On this plot, the wave directions are represented along the circumference of the circle and the wave periods along the radii. A wave direction of 90° represents a shore parallel direction. Therefore, the directions shown are relative to the shoreline, as opposed to true North. The color gradations represent the amount of energy for a given direction and frequency.

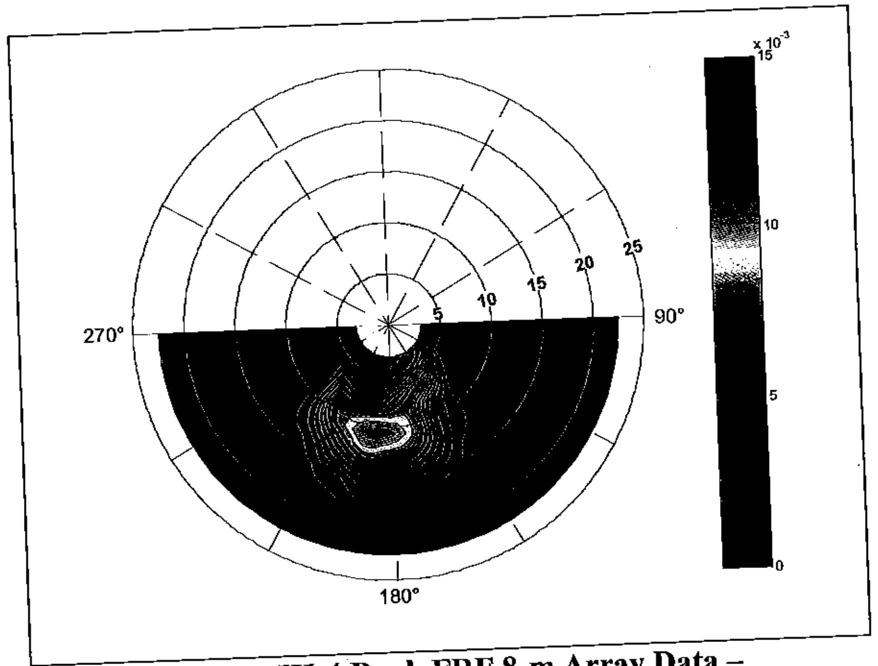


Figure III-4 Duck FRF 8-m Array Data – January Wave Energy Vs Period and Direction

From this plot the two wave trains can be distinguished, with the swell waves being the circle of energy between 180° and 225° with longer periods and the sea data being the smaller finger of energy between 150° to 180° with shorter periods. Based on the results for each month, the division for calculating sea and swell data was set at 5.5 seconds, such that all energy corresponding to periods less than or equal to 5.5 seconds was considered sea energy and all energy corresponding to periods greater than 5.5 seconds was considered swell energy. Having established this division, the sea and swell time series were computed as stated, by summing the total energy in each frequency bin (i.e., less than or equal to 5.5 sec or greater than 5.5 sec) and using an equation to calculate wave height based on the standard deviation of the sea surface elevations.

Since this data was nearshore data, the next step involved back-refracting the Duck wave data to deep water and comparing the "offshore" Duck data with other offshore measured and hindcast wave data gages which were near the Bay mouth and closer to the study area. The Duck data was back-refracted to the offshore using the straight and parallel method outlined in the USACE's Coastal Engineering Manual (CEM). The WIS Station 59 data set was selected for comparison because it had comparable data for an overlapping time period of measurement and was broken into two wave trains. Furthermore, the WIS hindcast data matched well with the measured data at the Chesapeake Light gage based on a percent exceedance analysis of the total significant wave heights at both gages during 1984-1995 **(Figure III-5)**.

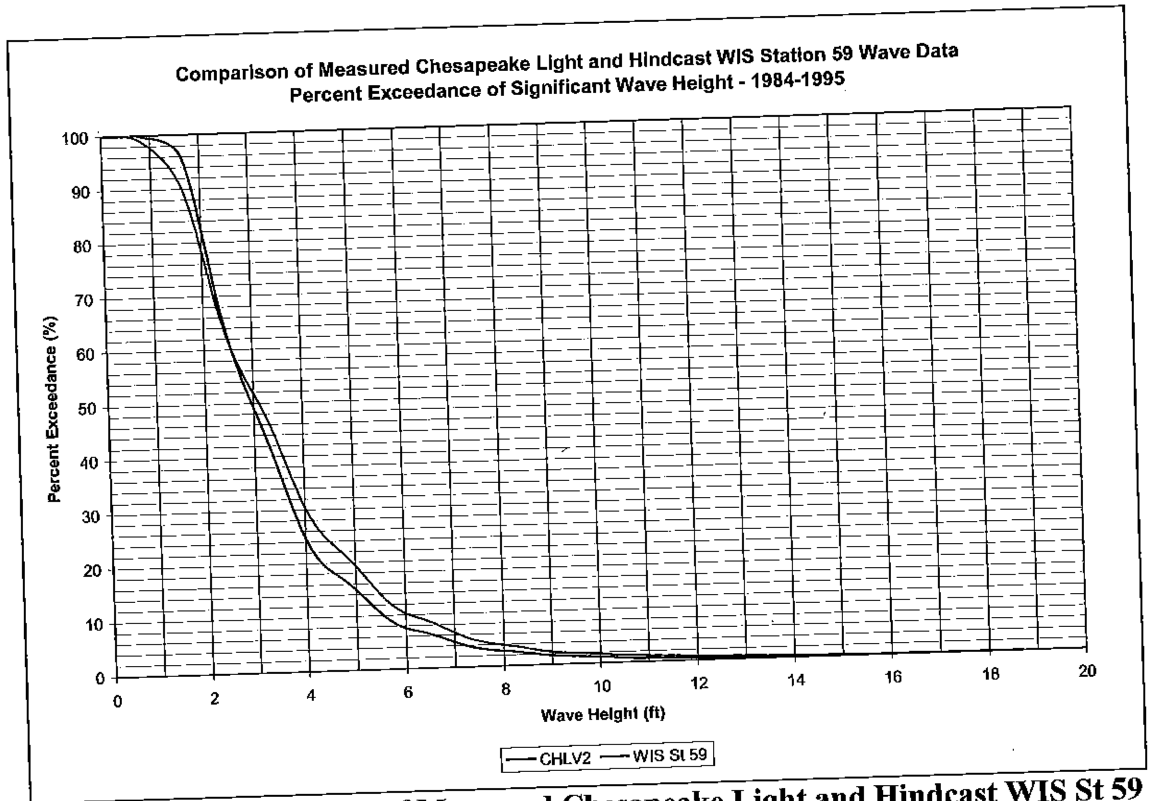


Figure III-5 Comparison of Measured Chesapeake Light and Hindcast WIS St 59 Wave Data

For comparative purposes, the "offshore" Duck sea and swell time series was then transformed (using the straight and parallel contour method) to the same water depth as the WIS Station 59 to allow for comparison with the WIS Station 59 data. As noted on **Table II-1**, the WIS data reported primary and secondary components within the wave data time series. These components do not correspond directly to sea and swell data, however, the sea and swell data can be discerned from these components if the division in wave period between the two wave trains is defined. Therefore, the same division of 5.5 seconds was used to create two time series of sea and swell from the WIS data. Finally, a comparison of the overlapping time period (1991-1995) between the WIS and "offshore" Duck data was performed to validate the use of the "offshore"

Duck data. **Figures III-6** and **III-7** show a comparison of the percent exceedance of wave heights for the sea and swell time series for both stations.

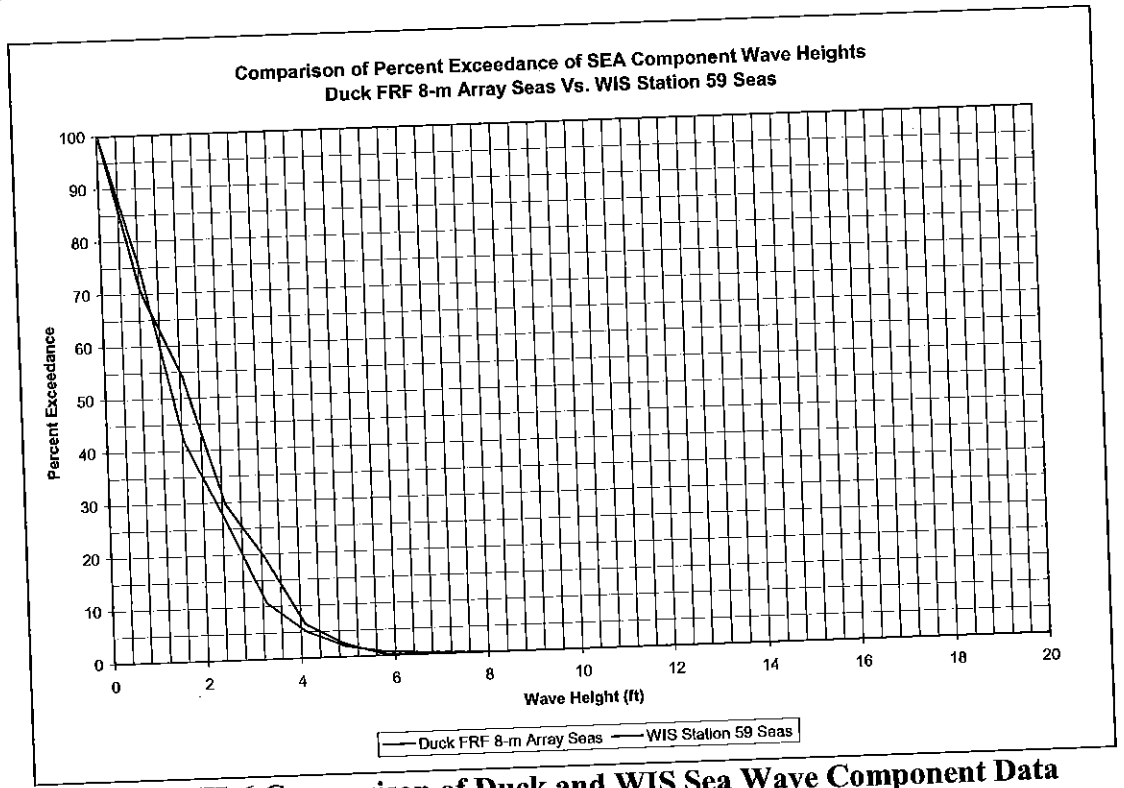


Figure III-6 Comparison of Duck and WIS Sea Wave Component Data

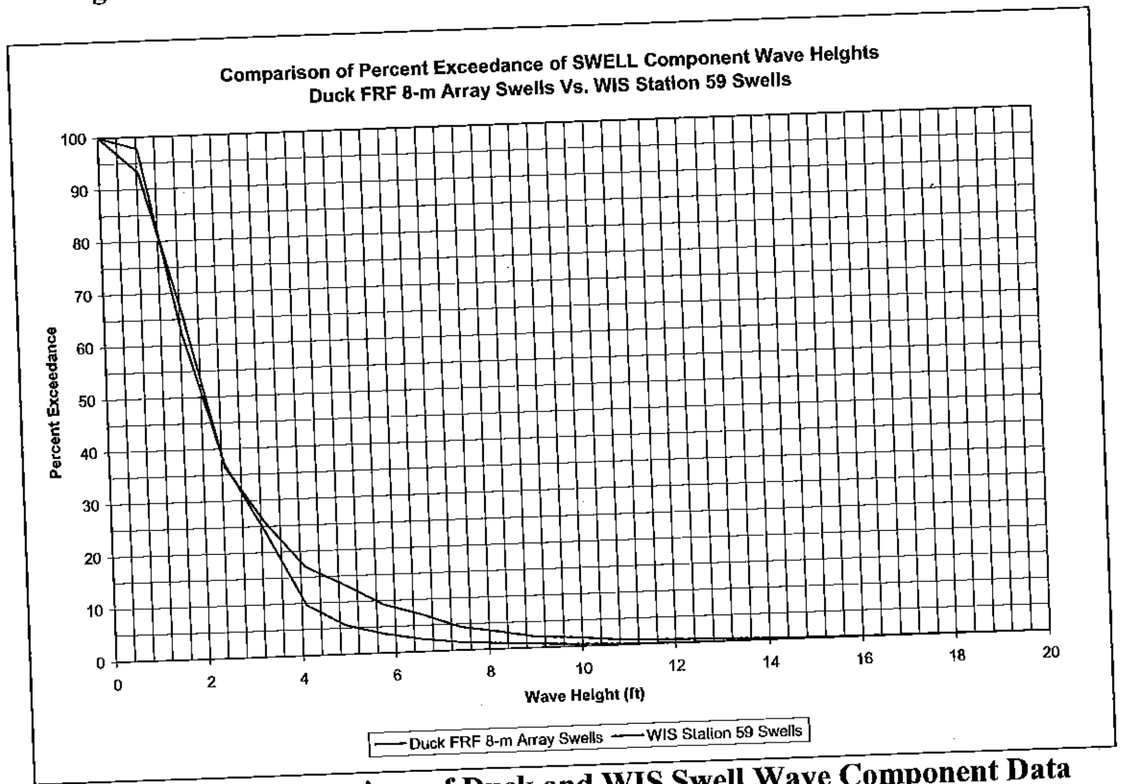


Figure III-7 Comparison of Duck and WIS Swell Wave Component Data

As shown in the above figures, the Duck data was consistent with the hindcast WIS data. This analysis validated the use of the generated sea and swell time series to represent offshore wave conditions which could now be transformed individually into the bay and to the nearshore site location.

The final offshore sea and swell time series spanned from 1991 to 2003 at 6 hour increments. Each wave component contained the significant wave height, peak wave period, and mean wave direction for each time step.

D. TRANSFORMATION OF DUCK WAVE DATA

To complete the required model inputs, the Duck sea and swell wave data needed to be transformed from the offshore location to a nearshore location indicative of -20 ft NAVD 88. Both wave trains were transformed using different methodologies. In general, the swell data was transformed by applying a refraction coefficient to the wave height and applying a refracted wave direction to transform the data to the nearshore location. The sea data was transformed using a fetch ratio procedure to factor the wave heights and periods from a deepwater location to the nearshore location. These procedures and the resulting finalized long-term wave data sets are discussed in more detail below.

1. Transformation of Swell Wave Data

As stated, the swell data was transformed to nearshore conditions using a refraction analysis. The refraction model utilized was the Danish Hydraulic Institute's (DHI) Mike 21 Nearshore Spectral Waves (NSW) model. This model simulates the growth, decay, and transformation of wind-generated waves and swell by taking into account the effects of refraction and shoaling due to varying depth, local wind generation and energy dissipation due to bottom friction and wave breaking. The model requires a base bathymetry grid as input, and a definition of a series of wave heights, wave directions, and wave periods to simulate. By running a constant unit wave height of 1 meter for the simulation along with a set of defined wave direction and wave period combinations, the NSW model output gives the refraction coefficient (K_r) and the refracted wave direction (RWD) which can be applied to an offshore wave height and direction for a point on the grid boundary to yield the refracted nearshore wave parameters.

For this application, the base bathymetry grid was generated from two combined sources:

- 30 meter digital elevation model (DEM) of the Chesapeake Bay bathymetry from NOAA – National Ocean Service (NOAA-NOS)
- Contours digitized from NOAA navigational charts
    - NOAA Chart No. 12207 "Cape Henry to Currituck Beach Light"
    - NOAA Chart No. 12221 "Chesapeake Bay Entrance"

The resulting bathymetry grid is shown in **Figure III-8**.

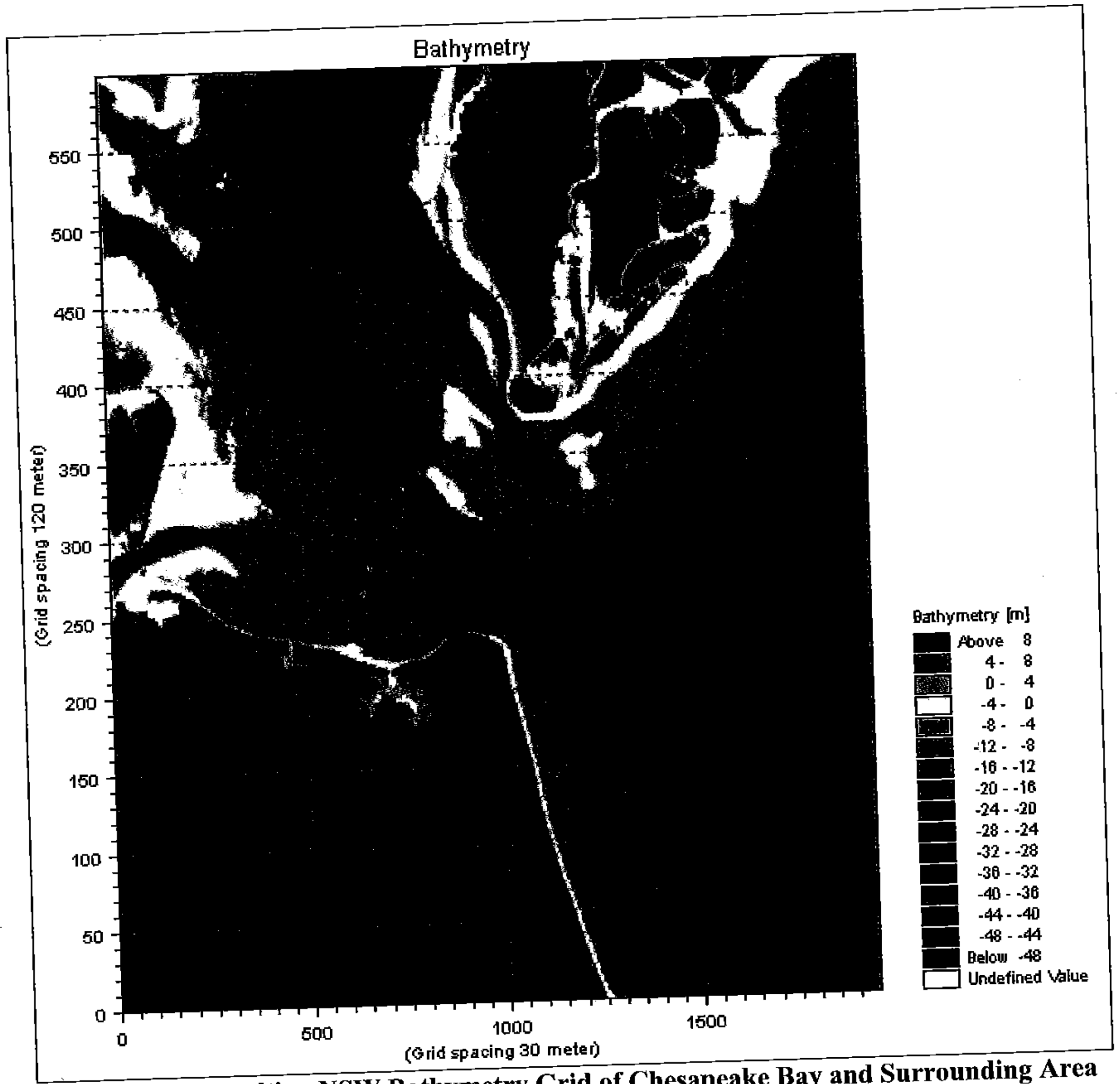


Figure III-8 Resulting NSW Bathymetry Grid of Chesapeake Bay and Surrounding Area (Note: Red Represents Land)

Along with the bathymetry model input, a number of input wave conditions were defined for the model runs. Again, a constant unit wave height of 1 m was used for all model runs. The wave directions modeled ranged from 60° to 120° at 5° increments. These wave directions are measured from North and are the angle from which a given wave approaches the model boundary. This range of wave directions was set based on which offshore wave direction could possibly enter the bay and ultimately reach the site. For this model, the boundary was the western side of the bathymetry grid. The wave periods modeled ranged from 4 seconds to 18 seconds at 1 second intervals. A model run was performed for every wave direction and wave period combination.

An example of the refraction coefficient output for the model run combination using a 80° wave direction and a 6 second wave period is shown in **Figure III-9**. Graphical results for a representative selection (wave periods = 4 sec, 8 sec, 12 sec, and 16 sec; wave directions = 60°, 70°, 80°, 90°, 100°, 110°, 120°) of the NSW model runs can be found in **Appendix F**.

Figure III-9 NSW Output – Wave Height Refraction Coefficients
(Note: White cells on grid represent areas of no data; Grey cells represent land)

The output from the NSW model is wave height in meters. However, since the model run involved a constant unit wave height of 1 meter, the output is equivalent to a refraction coefficient (K_r) if applied to a wave condition on the boundary. As shown in the figure, the wave heights are reduced as the swell waves enter the bay mouth and refract into shallower water. Please also note the difference in model results at the project site in relation to the other measured/hindcast wave stations shown in **Figure II-1**. These observed differences confirmed the need for this overall wave data analysis and transformation. Using the model results for all wave direction and wave period combinations, a grid point located at approximately -20 ft just offshore of East Ocean View (shown above) was specified for extracting the model output. The resulting wave height refraction coefficients and the refracted wave directions were extracted for all model runs at this point. **Tables III-2** and **III-3** show the resulting refraction coefficients (K_r) and refracted wave directions (RWD) for the modeled wave direction and wave period bins, respectively.

Table III-2 Wave Height Refraction Coefficients

Wave Height Refraction Coefficients (K_r)

	Wave Direction (degrees azimuth)												
Wave Period (sec)	60	65	70	75	80	85	90	95	100	105	110	115	120
6	0.28	0.31	0.34	0.35	0.31	0.30	0.28	0.26	0.23	0.20	0.16	0.13	0.11
7	0.25	0.26	0.26	0.26	0.22	0.22	0.21	0.20	0.19	0.17	0.15	0.13	0.11
8	0.21	0.21	0.21	0.20	0.18	0.18	0.18	0.17	0.16	0.15	0.13	0.12	0.10
9	0.19	0.19	0.18	0.18	0.16	0.16	0.16	0.16	0.15	0.14	0.13	0.11	0.10
10	0.17	0.17	0.17	0.17	0.15	0.16	0.16	0.16	0.15	0.14	0.13	0.11	0.10
11	0.17	0.17	0.16	0.16	0.15	0.15	0.16	0.15	0.15	0.14	0.13	0.11	0.10
12	0.16	0.16	0.16	0.16	0.15	0.15	0.16	0.15	0.15	0.14	0.13	0.11	0.10
13	0.16	0.16	0.16	0.16	0.15	0.15	0.16	0.15	0.15	0.14	0.13	0.11	0.10
14	0.16	0.16	0.16	0.16	0.15	0.15	0.16	0.15	0.15	0.14	0.13	0.11	0.10
15	0.16	0.16	0.16	0.16	0.15	0.16	0.16	0.16	0.15	0.14	0.13	0.12	0.10
16	0.16	0.16	0.16	0.16	0.15	0.16	0.16	0.16	0.15	0.14	0.13	0.12	0.10
17	0.16	0.16	0.16	0.16	0.15	0.16	0.16	0.16	0.15	0.14	0.13	0.12	0.10
18	0.16	0.16	0.16	0.16	0.15	0.16	0.16	0.16	0.15	0.14	0.13	0.12	0.11

Table III-3 Refracted Wave Directions

Refracted Wave Direction (RWD)

	Wave Direction (degrees azimuth)												
Wave Period (sec)	60	65	70	75	80	85	90	95	100	105	110	115	120
6	66.12	67.64	68.80	69.47	67.16	66.81	66.37	65.99	65.66	65.27	64.71	63.96	62.97
7	65.08	65.64	65.96	65.99	62.96	62.33	61.75	61.39	61.18	61.03	60.89	60.72	60.45
8	62.84	62.64	62.28	61.76	58.83	58.21	57.65	57.22	56.90	56.66	56.48	56.31	56.05
9	60.29	59.64	58.93	58.26	55.85	55.40	54.93	54.49	54.11	53.80	53.54	53.24	52.82
10	58.26	57.47	56.71	56.10	54.03	53.70	53.30	52.89	52.51	52.19	51.89	51.50	50.96
11	56.85	56.06	55.36	54.83	52.93	52.66	52.31	51.94	51.59	51.28	50.95	50.52	49.92
12	55.90	55.15	54.51	54.06	52.22	52.00	51.69	51.35	51.02	50.73	50.40	49.94	49.31
13	55.24	54.54	53.95	53.55	51.75	51.55	51.27	50.96	50.66	50.37	50.04	49.57	48.93
14	54.77	54.11	53.57	53.20	51.42	51.24	50.98	50.69	50.40	50.13	49.80	49.32	48.68
15	54.42	53.79	53.28	52.94	51.18	51.01	50.77	50.49	50.22	49.96	49.62	49.15	48.50
16	54.15	53.55	53.07	52.75	50.99	50.84	50.61	50.35	50.09	49.83	49.50	49.02	48.38
17	53.95	53.37	52.91	52.60	50.85	50.70	50.48	50.23	49.98	49.73	49.40	48.92	48.28
18	53.78	53.22	52.78	52.48	50.74	50.59	50.38	50.14	49.90	49.65	49.32	48.84	48.20

Using the above results, the offshore swell data was then refracted to nearshore conditions by multiplying the offshore wave height by the corresponding refraction coefficient defined by the offshore period and direction. To determine the appropriate refraction coefficient, the offshore wave directions and wave periods were organized into 5 degree and 1 second bins respectively. For a given time step with an offshore wave direction of 82° and wave period of 7.3 seconds, the corresponding model output for the 80° and 7 second model run was applied. The wave heights were reduced by the defined refraction coefficient and a new refracted wave direction was defined. The wave periods did not change from the offshore definition.

Finally, there was missing data for short portions of the 1991-2003 Duck swell data time series. To develop a continuous time series, these missing data values were replaced with the previous measured values. For the purposes of the modeling tasks in this study, this did not seem unreasonable given that the wave conditions did not vary significantly under typical conditions.

This procedure resulted in a final nearshore swell wave time series. **Figure III-10** shows a statistical summary of the nearshore wave heights, wave directions, and wave periods, for the complete swell time series spanning 1991-2003. On these graphs, the x-axis represents the bins for each wave parameter (direction, period, and height) and the y-axis represents the percent occurrence of each defined bin in the overall swell wave data set.

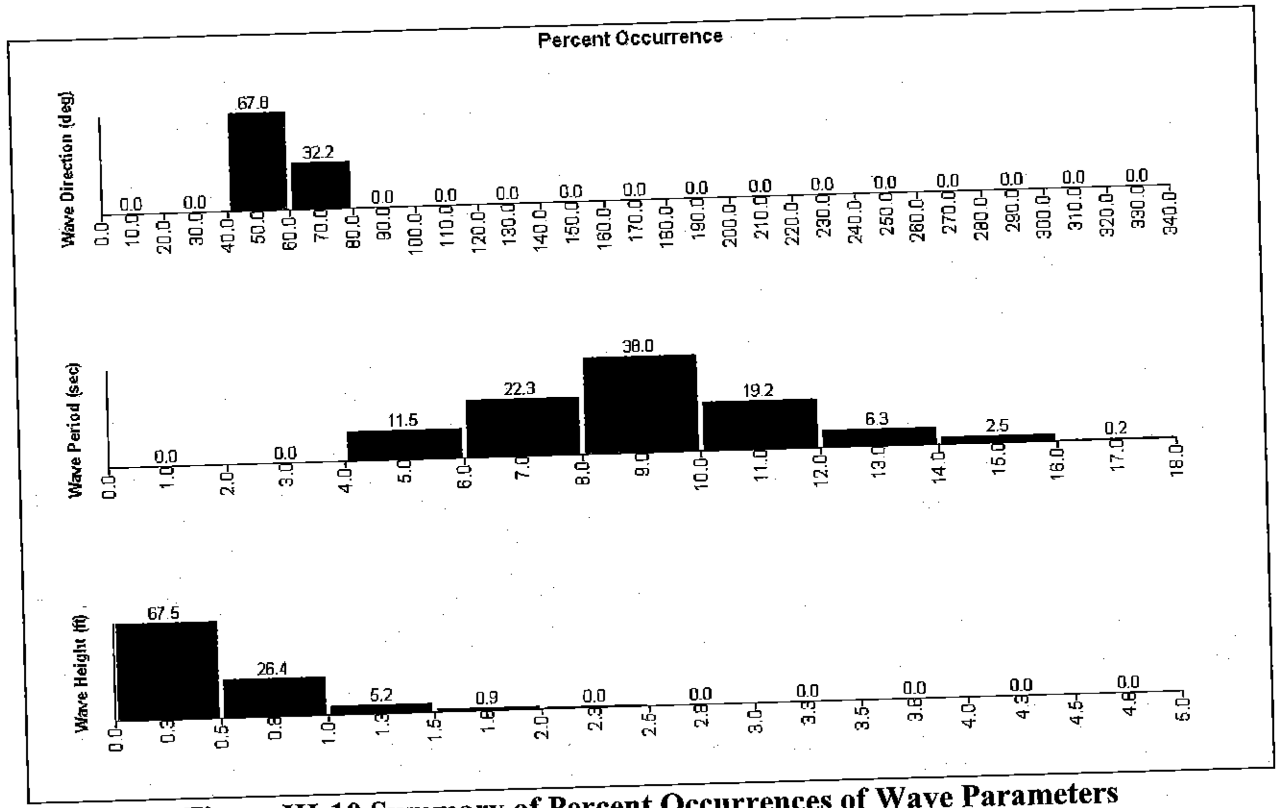


Figure III-10 Summary of Percent Occurrences of Wave Parameters in the Final Nearshore Swell Wave Data Time Series

As expected, a majority of the swell waves arrive at the study area from the Northeast direction and are not very steep at the site location. Please recall that the swell wave component represents only one portion of the combined waves impacting the site. The transformation of the sea wave data had yet to be completed.

2. *Transformation of Sea Wave Data*

Sea waves are characterized as locally generated wind waves. To transform the sea data time series from the Duck FRF site to the nearshore location at the study area, a fetch ratio analysis was performed. The fetch length for a given location represents the area over which ocean waves are generated by wind. A fetch length is defined by assuming a wind direction and calculating the length from the point of interest to landfall in the given wind direction. If for a given direction, the fetch length does not hit landfall, the sea generated waves are considered fetch-limited.

To determine the fetch ratios between the Duck FRF site and the study area, the fetch length for wind directions ranging from 315° to 0° and 0° to 45° (wind directions from North) were calculated for the Duck FRF site point and the nearshore model grid point used in the swell refraction analysis. **Figure III-11** depicts the fetch lines drawn for these wind directions at both sites.

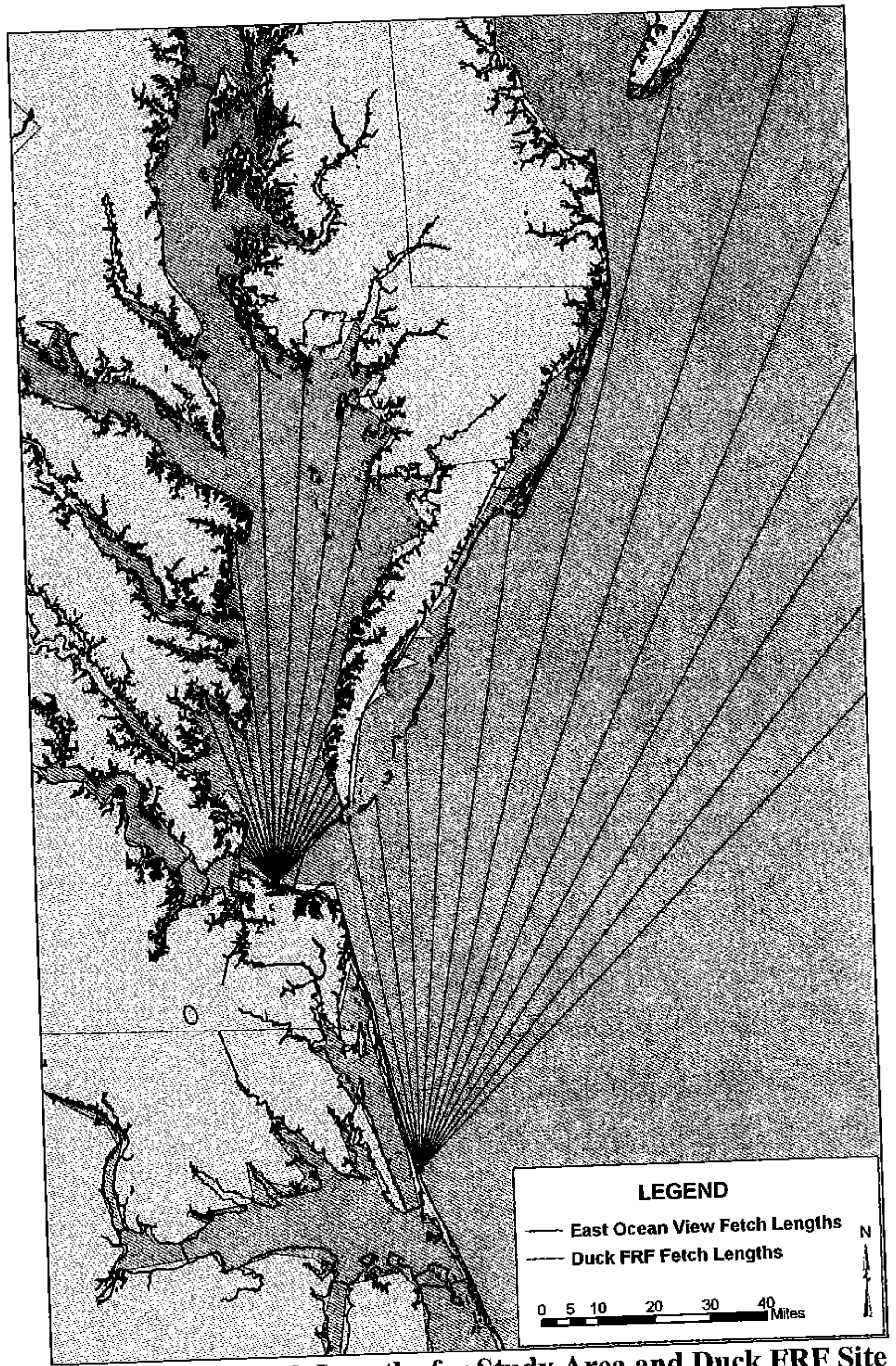


Figure III-11 Fetch Lengths for Study Area and Duck FRF Site

As shown on the figure, many of the fetch distances, particularly from the Duck site, were significantly long. It was not reasonable to assume that wind generated waves would travel over these lengths. Therefore, a fetch limiting distance was defined. Using the range of wave periods measured in the sea wave data time series, a maximum fetch length of 30 miles for a fully arisen sea was estimated based on guidance provided in the USACE CEM. Therefore, any fetch lengths determined which exceeded this distance at either site were reduced to 30 miles. This resulted in more realistic fetch ratios in generating the nearshore sea wave time series. Using the final fetch distances, the fetch ratios to apply to wave heights and wave periods were calculated using the following equations. Again, these equations were derived based on equations provided in the USACE's CEM.

Equation 1 – Wave Height Calculation

$$\frac{H_{site}}{H_{duck}} = \frac{F_{site}^{\frac{1}{2}}}{F_{duck}^{\frac{1}{2}}}$$ where H = wave height (ft) and F = fetch length (ft)

Equation 2 – Wave Period Calculation

$$\frac{T_{site}}{T_{duck}} = \frac{F_{site}^{\frac{1}{3}}}{F_{duck}^{\frac{1}{3}}}$$ where T = wave period and F = fetch length (ft)

The measured fetch distances and final fetch factors for the study area and Duck FRF sites are summarized in **Table III-4**.

Table III-4 Summary of Fetch Ratio Calculations

Wave Direction (degrees azimuth)	EOV - Fetch Distance (miles)	DUCK - Fetch Distance (miles)	Fetch Ratio - Wave Height -	Fetch Ratio - Wave Period -
45	30.00	30.00	1.00	0.57
40	16.50	30.00	0.74	0.46
35	18.57	30.00	0.79	0.48
30	19.16	30.00	0.80	0.49
25	20.56	30.00	0.83	0.50
20	22.97	30.00	0.87	0.52
15	28.00	30.00	0.97	0.55
10	30.00	30.00	1.00	0.57
5	30.00	30.00	1.00	0.57
0	30.00	30.00	1.00	0.57
355	30.00	30.00	1.00	0.57
350	30.00	30.00	1.00	0.57
345	26.70	30.00	0.94	0.55
340	30.00	25.78	1.08	0.61
335	30.00	11.27	1.63	0.93
330	11.20	5.22	1.46	0.98
325	10.32	2.79	1.92	1.30
320	9.62	1.47	2.56	1.75
315	9.07	1.19	2.76	1.91

As done in the swell data transformation, the sea wave time series data at Duck was summarized into bins defined by the wave direction at a given time step and the corresponding fetch factors were applied to the Duck wave heights and wave periods. For those time steps having measured wave directions within the bins defined above, the wave directions were not adjusted from the measured direction in the Duck sea time series. There was also some amount of sea wave energy which arrived from the mouth of the Bay (60° to 120°). For any waves in the Duck sea time series which had wave directions between 60° and 120°, the resulting refraction coefficients (K_r and RWD) defined from the NSW model output (applied to the swell time series) for wave periods ranging from 4 seconds to 6 seconds were applied to the offshore Duck wave heights and wave directions.

Having applied all of the appropriate factors to transform the sea data from the Duck site to the study area, a resulting nearshore sea wave time series spanning 1991-2003 was generated. As with the swell data, there were short periods of missing data over the entire sea data time series. To develop a continuous time series, these missing data values were replaced with the previous measured values (as with the swell data). **Figure III-12** shows a statistical summary of the nearshore wave heights, wave directions, and wave periods, for the complete sea time series. On these graphs, the x-axis represents the bins for each wave parameter (direction, period, and height) and the y-axis represents the percent occurrence of each defined bin in the overall sea wave data set.

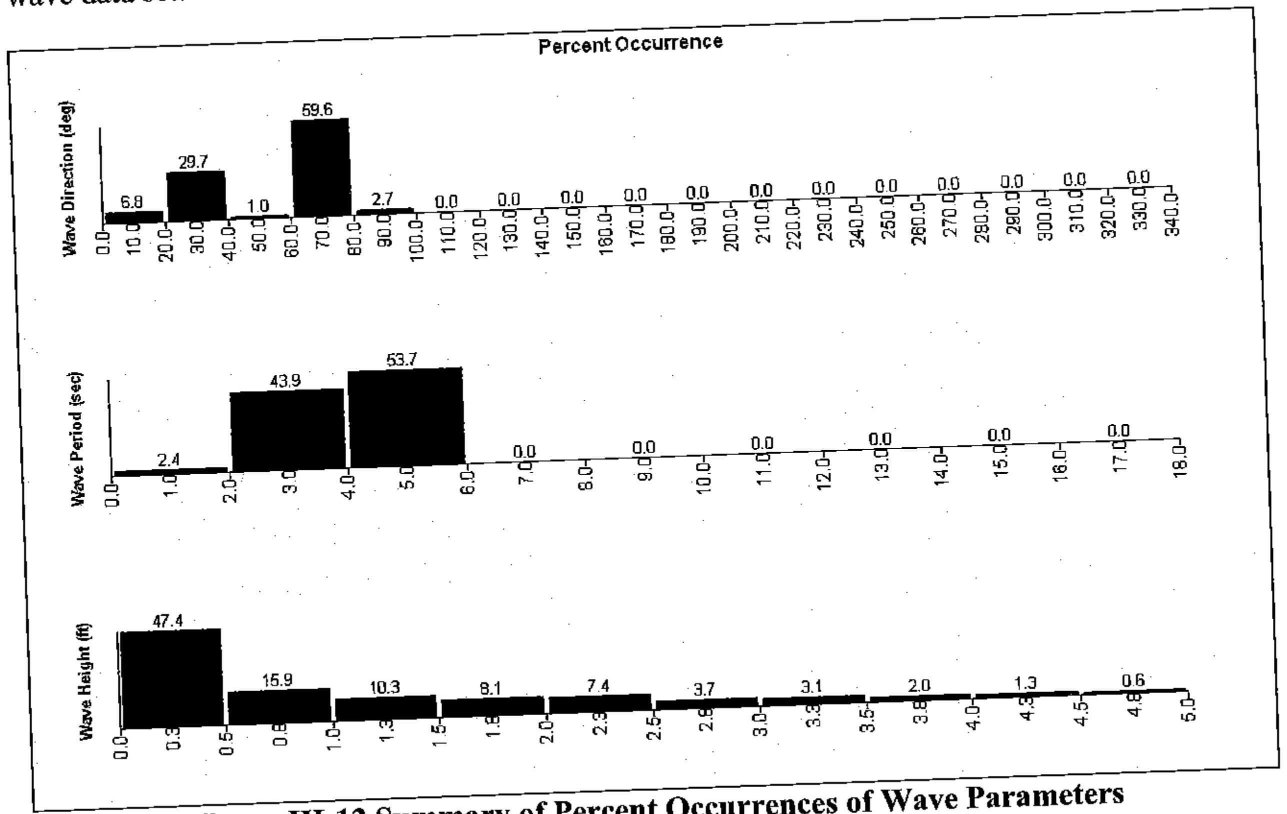


Figure III-12 Summary of Percent Occurrences of Wave Parameters in the Final Nearshore <u>Sea</u> Wave Data Time Series

In summary, the transformed sea and swell data was determined to be the most suitable of the available data sets given the specific goals of the coastal modeling applications. Most crucial to this study, the measured Duck data allowed for the generation of the bimodal sea and swell wave energy components. The resulting transformed data was considered more applicable than the measured data within the Bay (Boon TSL or USACE OCTI Hindcast data) because it was a consistent long time series which exhibited both long-term typical conditions and storm events. Nonetheless, M&N strongly suggests that the City consider installing wave gages in order to acquire more accurate wave data which would allow for greater confidence and efficiency in the decisions of future shoreline protection projects. Secondly, the transformations resulted in wave data that included higher waves than the relative measurements at Dr. Boon's TSL gage, which may have underestimated the general wave climate if applied to the East Ocean View study area. This point was further verified by the significant difference in the resulting refraction coefficients from the NSW model between the nearshore study area location and the locations of the TSL and USACE data stations (**Figure III-9**). Having developed the applicable nearshore sea and swell component wave time series data sets, the coastal modeling tasks were initiated.

IV. SBEACH MODEL

SBEACH (Storm-induced BEAch CHange) is an empirically based numerical simulation model which was developed at the U.S. Army Corps of Engineers (USACE) Waterways Experiment Station (WES) Coastal & Hydraulics Laboratory (CHL). The purpose of the model is to calculate two-dimensional, cross-shore beach, berm, and dune erosion under single-storm surge, wave, and wind action. The SBEACH model is based on a fundamental assumption that profile change is produced only by cross-shore processes. Therefore, longshore processes are considered uniform and neglected in calculating profile change. The cross-shore sediment transport processes are governed by empirical equations defined for four distinct zones in the nearshore: swash, broken wave, breaker transition, and prebreaking. For a more detailed description of the sediment transport mechanisms governing SBEACH, the reader is referred to a series of USACE reports published on the model (Larson and Krauss, 1989, Larson et al., 1990, Rosati et al., 1993).

The most recent version of SBEACH, released in 2002, operates under the Coastal Engineering Design and Analysis System (CEDAS), a suite of tools developed by Veri-Tech, based on various numerical models and codes developed at WES. The CEDAS suite also includes BMAP (Beach Morphology Analysis Package) and GENESIS (Generalized Model for Simulating Shoreline Change), tools which were also utilized in this study.

The SBEACH model has potential for many applications in the coastal environment, including evaluation of design beaches for erosion and/or flood protection, evaluation of short-term beach fill performance, and preliminary input for economic analyses of beach alternatives.

The main inputs to the SBEACH model include:

- Profile Data – two-dimensional description of the shoreline extending from offshore to a landward point of interest,
- Sediment Data - characterization of the average sediment size and,
- Storm Data – time dependent description of water elevation, waves, and winds (if available).

A. SCOPE OF SBEACH MODELING

The scope of the SBEACH modeling for the East Ocean View site involved evaluating the immediate cross-shore loss of sand in the berm and/or dune for a one year time period. Following this time period, it was expected that the beach would reach an equilibrium profile position, which would serve as the basis for the long-term shoreline evolution modeling (GENESIS). Individual SBEACH models of a particular site and duration can be calibrated if pre- and post- measured profiles are available. The model calibration parameters include a number of sediment transport characteristics and other beach characteristics (avalanche angle, landward surf zone depth) that influence sediment transport.

To establish the appropriate model parameters, the SBEACH model was calibrated using historical profile data and coinciding wave and tide data. Once the model was calibrated, an existing conditions SBEACH model was generated using the established calibration coefficients and a representative sample of the post-construction beach profiles. The resulting profile positions after this 1-year model simulation were used to generate the equilibrium shoreline position for the GENESIS model.

Finally, a separate SBEACH modeling task involved simulating the impact of Hurricane Isabel on the equilibrium profile. Again, a calibration model was developed first using pre-storm (June 2003) and post-storm (September 2003) surveys to establish the appropriate model coefficients. Then the storm was run on the representative equilibrium profiles which were the output of the 1-year SBEACH simulation.

B. SBEACH MODEL – ONE-YEAR ANALYSIS

As stated previously, the initial SBEACH modeling task involved simulating the impact of one year of wave and water level conditions on the post-nourished beach profile to determine the equilibrium profile. The equilibrium profile aided in the generation of the initial shoreline used in the GENESIS long-term modeling.

1. Model Calibration

A calibration model was developed to determine the appropriate SBEACH coefficients for simulating the 1-year impact analysis. The duration of the SBEACH model was dependent on the dates of available measured beach profile data and available wave data. A subset of the nearshore wave time series sea and swell components (1991-2003) were used for defining the required wave inputs to the model.

Based on the available survey data, a duration spanning October 1999 to July 2000 was selected for the SBEACH calibration model. These dates were also chosen because they did not overlap with the construction or presence of the offshore breakwaters at the site (completed in August 2000 and November 2001). This would allow for a true estimation of the equilibrium profile shape without structural effects.

a) Profile Data (1-Year Calibration Model)

The profile data was obtained from the City of Norfolk surveys. A single profile located just approximately 1200 ft west of Little Creek Jetty was selected for the calibration model. The survey points for this profile were extracted from AutoCAD for both the October 1999 and July 2000 survey dates and 2D cross-sectional profiles were generated. The measured October 1999 profile became the initial beach profile for the SBEACH model input. The measured July 2000 profile was also loaded into the model to serve as a reference profile position for the model calibration. **Figure IV-1** shows a comparison of these profiles. The first point on both profiles is artificial and was added because the measured data extended only just over the crest of the dune.

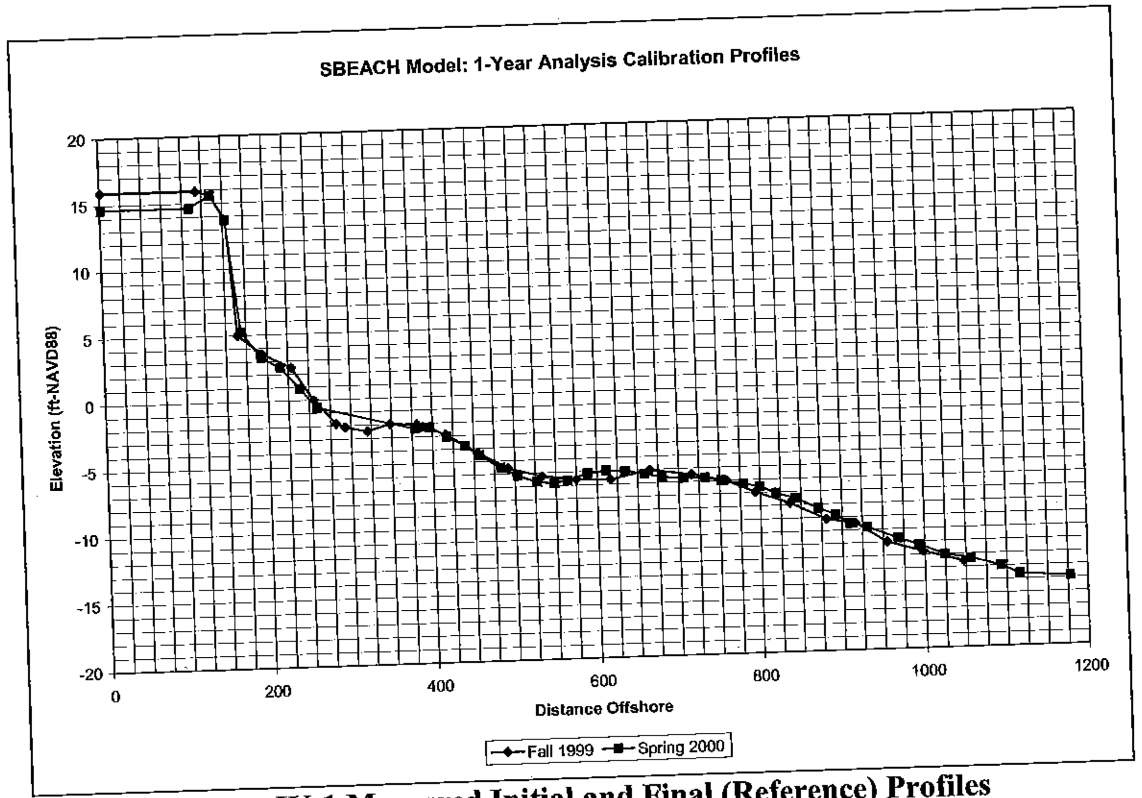


Figure IV-1 Measured Initial and Final (Reference) Profiles for SBEACH Calibration Model

b) Storm Data (1-Year Calibration Model)

Typical storm data input for SBEACH includes storm hydrographs of total water elevation, wave conditions, and wind conditions. For this analysis, the simulation involved a longer time series representing continuous typical water and wave action. Wind data is an optional input in the SBEACH model, but was not used in this application, since typical wind conditions are not likely to result in waves causing significant loss of sediment from the berm or dune.

The wave time series included significant wave height and peak wave period extracted from the nearshore sea and swell wave time series for a duration spanning from late September 1999 to late May 2000. The dates of the wave duration did not match exactly with the measured survey dates. This subset of data was selected because there were some breaks in the wave data during June and July 2000, where constant wave conditions were assumed. Therefore, the wave data input was shifted slightly from the survey dates to achieve more typical wave conditions throughout the model duration.

SBEACH only allows the user to input a single wave time series. Therefore, for this analysis, a combined single wave time series was generated which consisted of alternating sea and swell data records with a 3-hour time step. **Figure IV-2** and **IV-3** show the resulting combined (sea + swell) significant wave height and peak wave period time series, respectively.

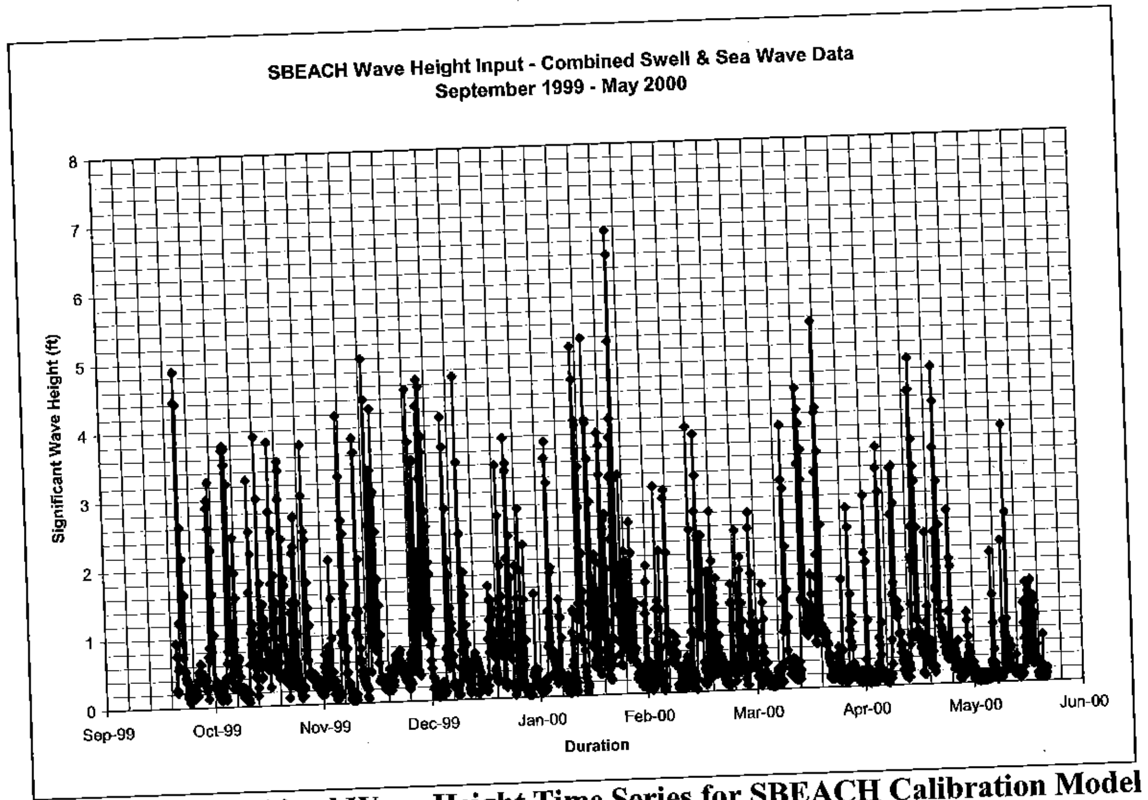


Figure IV-2 Combined Wave Height Time Series for SBEACH Calibration Model

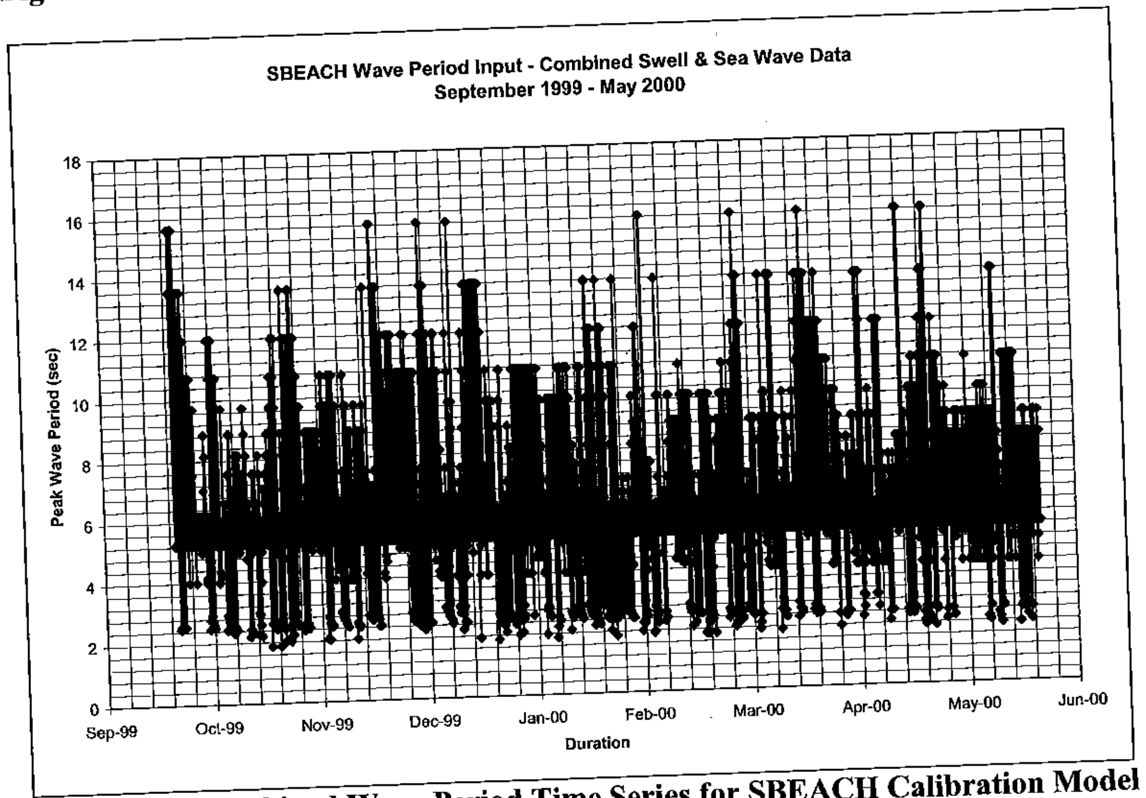


Figure IV-3 Combined Wave Period Time Series for SBEACH Calibration Model

The water level input for the SBEACH calibration model was obtained from the NOAA Chesapeake Bay Bridge tide station. The measured tide data was extracted at a 3-hour time step for the model duration from October 1999 to July 2000. The elevations were converted to the NAVD 88 datum. **Figure IV-4** shows the resulting measured water level data used for the SBEACH calibration model.

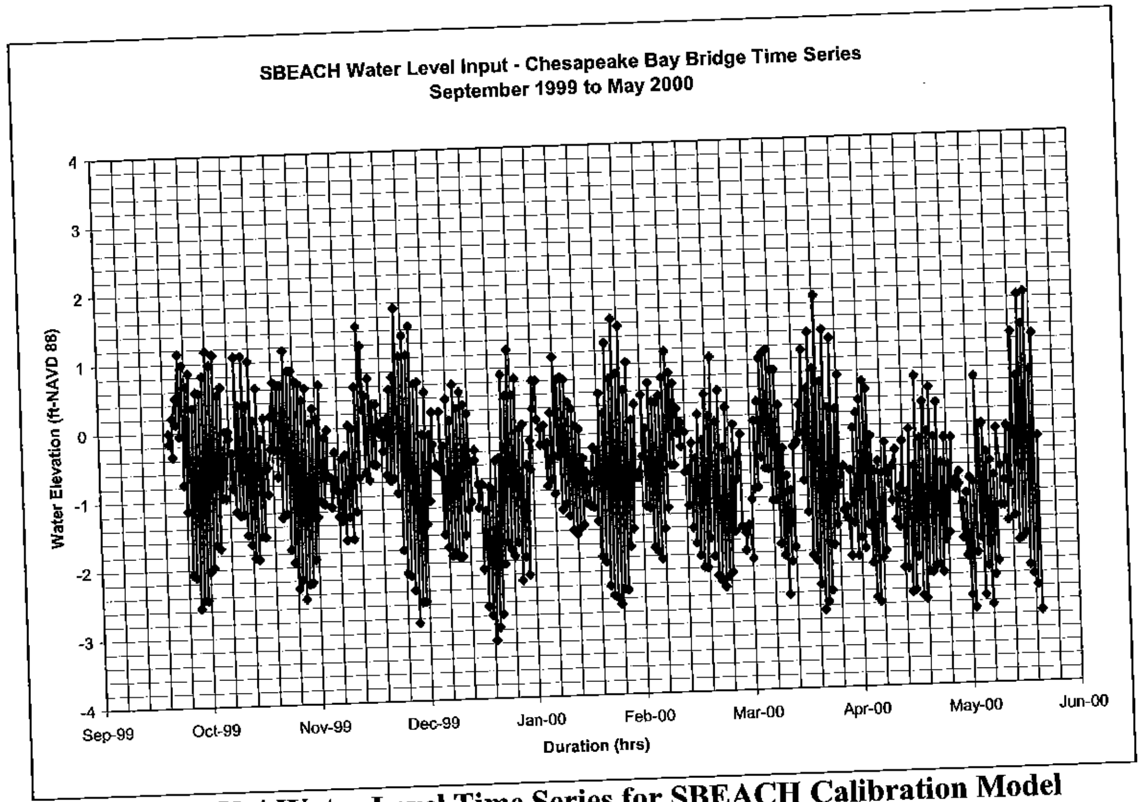


Figure IV-4 Water Level Time Series for SBEACH Calibration Model

c) Sediment Data (1-Year Calibration Model)

As described in **Section II** of this report, the effective grain size used for all calibration models was consistent with the sediment data collected prior to the beach nourishment project and also prior to Hurricane Isabel. For this analysis, an effective grain size of 0.23 mm was assumed for the model profile.

d) Model Calibration Coefficients (1-Year Calibration Model)

The model calibration coefficients consisted of sediment transport characteristics and other beach characteristics (avalanche angle, landward surf zone depth) that influence sediment transport. The initial model was run using the default values defined for these parameters. Then, each parameter was adjusted individually, within the recommended range, to determine its influence on the model output. The appropriate model coefficients were determined by comparing the SBEACH final output profile with the measured July 2000 profile.

After running a number of model scenarios, it was determined that the following model parameters yielded the most accurate final profile.

- Transport Rate Coefficient, K (m^4/N) = 2.5 x 10-7,
- Coefficient for Slope-Dependent Term, Eps (m2/s) = 0.0001,
- Transport Rate Decay Coefficient Multiplier = 0.5,
- Landward Surf Zone Depth = 1.6, and
- Avalanche Angle = 30°

e) <u>Model Output (1-Year Calibration Model)</u>

A comparison of the final model profile and the measured final profile for July 2000 is shown in **Figure IV-5**. As shown, the final SBEACH profile matches well with the measured July 2000 profile. Along the portion of the profile that SBEACH simulates cross-shore change (i.e. berm and dune), the final profile is consistent with the minor loss of sediment that occurred over the time period of the calibration. Please note that all of the SBEACH model runs tested showed the same behavior as shown at the offshore bar at Station 4+00, so it was decided that it was more important to match the profile above an elevation of 0-ft NAVD 88. Based on the results of this analysis, it was reasonable to assume that these calibration coefficients could be applied in a model of the existing conditions (post-nourishment) to determine the expected 1-year loss and equilibrium profile position.

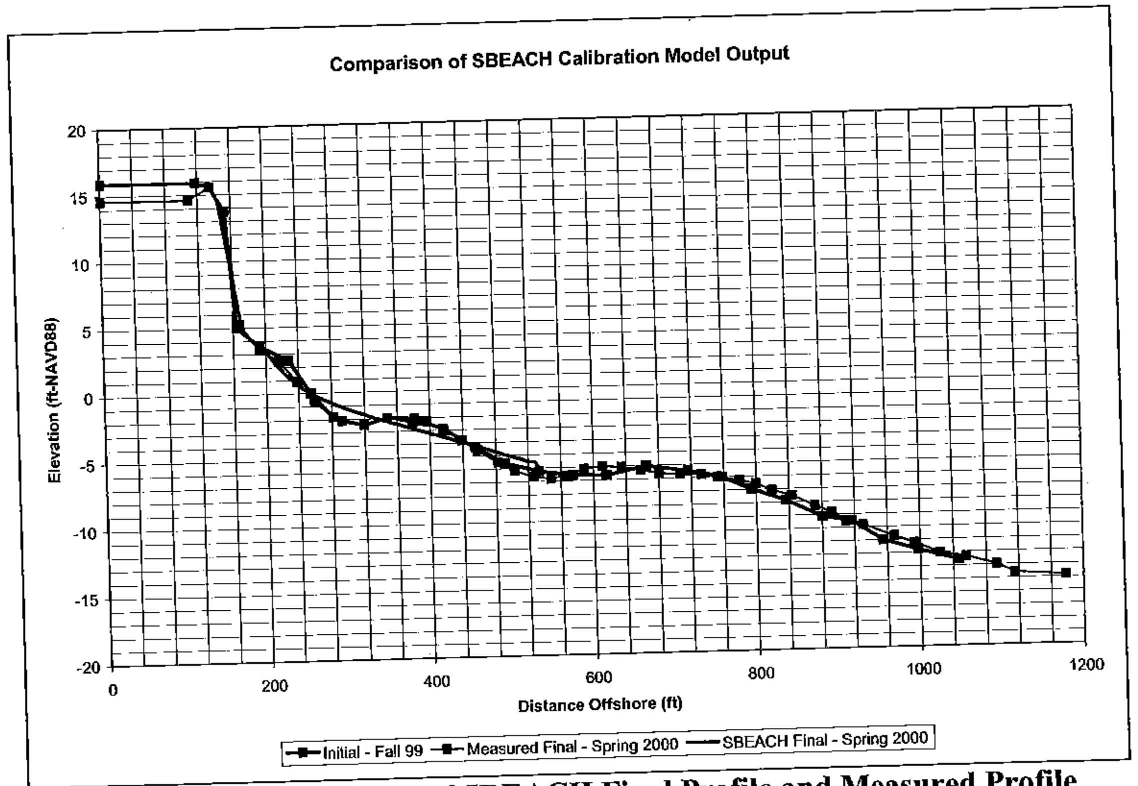


Figure IV-5 Comparison of SBEACH Final Profile and Measured Profile for Calibration Model

2. *Existing Conditions Model*

With the SBEACH calibration coefficients now set, the existing conditions model was developed to estimate the initial cross-shore beach change that the East Ocean View study area would be expected to experience following a year of typical wave and water level impacts. The model was developed using the defined calibration coefficients, the post-nourishment beach surveys, and a portion of the established nearshore swell and sea wave data time series. The duration of the simulation was one year, beginning at the date of the post-construction surveys (Dec 2003), and spanning until approximately December 2004.

a) Profile Data (1-Year Existing Conditions Model)

The input profiles for the SBEACH model were generated using the post-nourishment survey data collected by Waterway Surveys & Engineering in November-December 2003. To simplify the model runs, a representative set of survey cross-sections were selected as model input. Individual cross-sections were selected to represent typical profiles for sections of the shoreline. Additionally, consecutive cross-sections were selected where a unique condition existed on the shoreline, such as stations located behind and between breakwaters, stations at the Kois property, and stations at the stormwater outfall pipe. The stationing of the survey data and resulting profiles coincided with the stationing represented on the beach fill project. The selected profile stations are listed in **Table IV-1** and are shown on **Figure IV-6**.

Table IV-1 List of Selected Model Profile Stations (From Post-Nourishment Survey Cross-Sections)

Survey Cross-Section Station
St 4+00
St 8+00
St 9+00 – Kois Property
St 12+00
St 18+00 – between breakwaters
St 19+00 – behind breakwater
St 28+00 – behind breakwater
St 31+00 – behind breakwater
St 33+00 – outfall pipe location
St 34+00 - outfall pipe location
St 35+00 - outfall pipe location
St 38+00 - behind breakwater
St 41+00
St 45+00
St 49+00
St 52+00

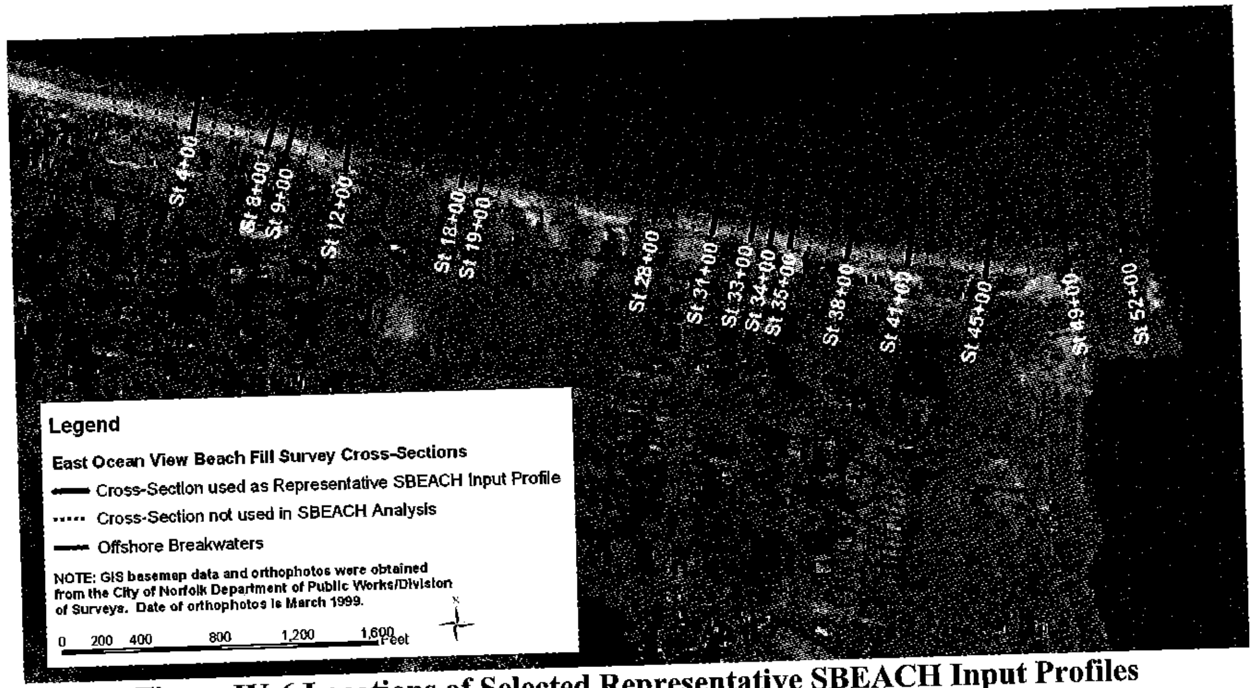


Figure IV-6 Locations of Selected Representative SBEACH Input Profiles

The post-fill surveys only extended over the dune and to a position offshore at approximately -6 ft NAVD 88. Since SBEACH requires a profile extending offshore to the expected depth of closure, the remainder of the profile on the landward and seaward sides were generated by splicing the post-construction surveys with the June 2003 survey data. This was a reasonable procedure given the fact that SBEACH simulates change in the berm and dune region of the profile and that the offshore profile can be expected to stay fairly consistent over time. In addition, for those profiles which crossed breakwaters, the breakwater profile had to be entered as a hard bottom portion of the profile in the SBEACH model. **Figure IV-7** shows the SBEACH input profile for Station 4+00. All of the final beach and bathymetric profiles included in this SBEACH analysis are shown in **Appendix G.**

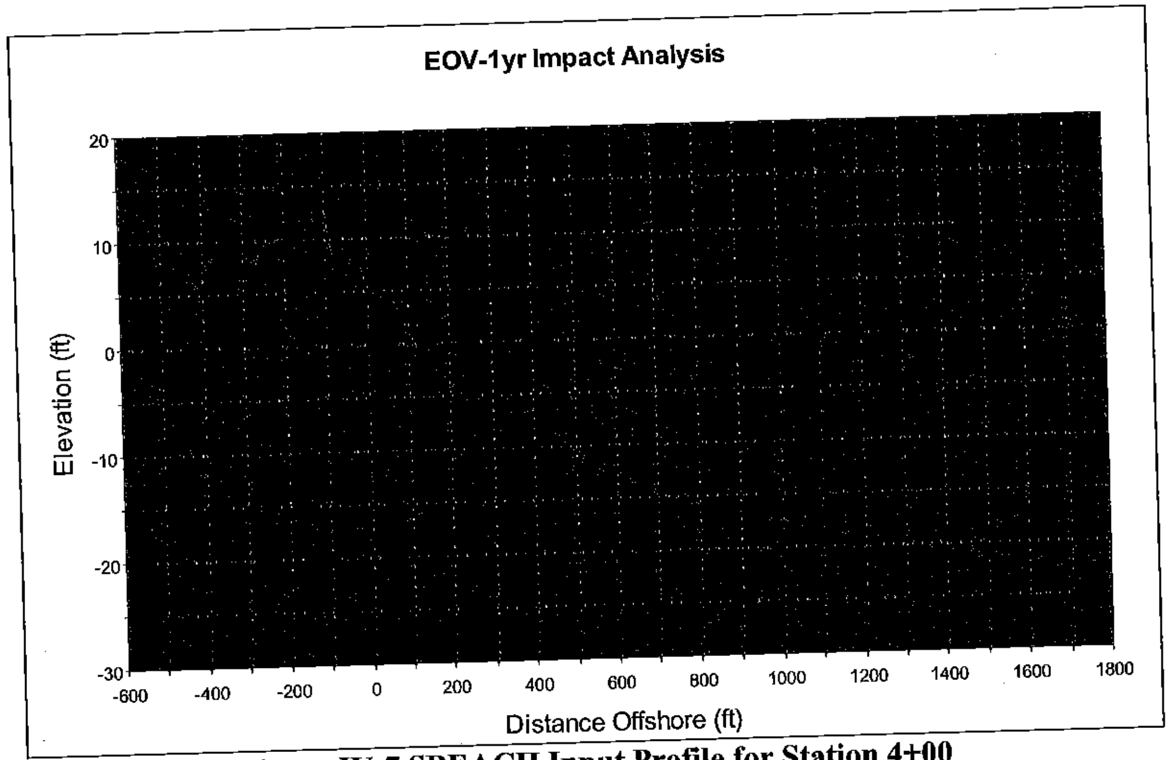


Figure IV-7 SBEACH Input Profile for Station 4+00

b) Storm Data (1-Year Existing Conditions Model)

As stated previously, the duration of the SBEACH existing, post-fill conditions 1-year model analysis extended for a year beginning at the date of the post-construction surveys (December 2003). To develop the input wave time series for the SBEACH model, a representative historical year of nearshore wave data was selected for the simulation. The selected time series extended from November 2000 through October 2001 and was selected because it had consistent measurements with a number of apparent storm events. As done for the calibration model, a combined time series was generated which consisted of alternating swell and sea records, so that both wave components were simulated. **Figures IV-8** and **IV-9** show the resulting, combined wave height and wave period time series, respectively.

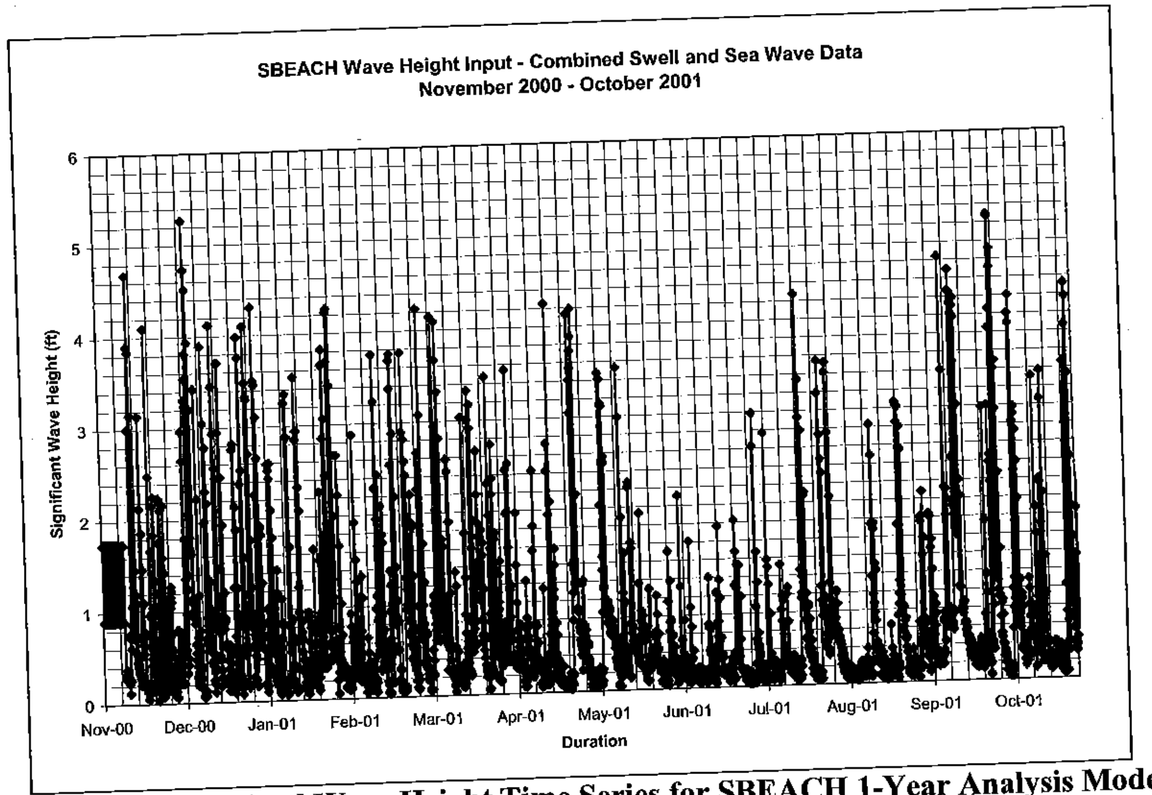


Figure IV-8 Combined Wave Height Time Series for SBEACH 1-Year Analysis Model

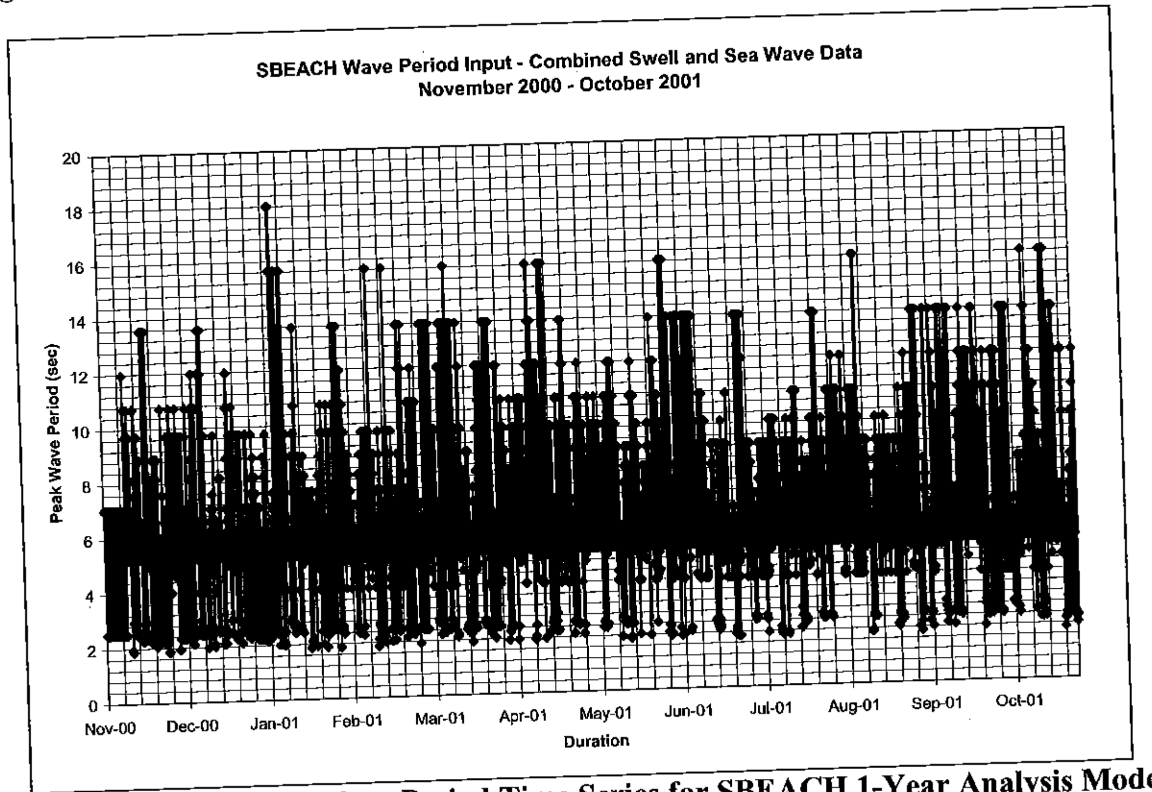


Figure IV-9 Combined Wave Period Time Series for SBEACH 1-Year Analysis Model

Again, the water level data input came from the measured data at the Chesapeake Bay Bridge Tunnel gage. This data was extracted for the same time period (Nov 2000 through Oct 2001) as the wave data and at 3-hour time steps. The water levels were converted to the NAVD 88 tidal datum. **Figure IV-10** shows the resulting water level time series used in the existing conditions 1-year SBEACH model.

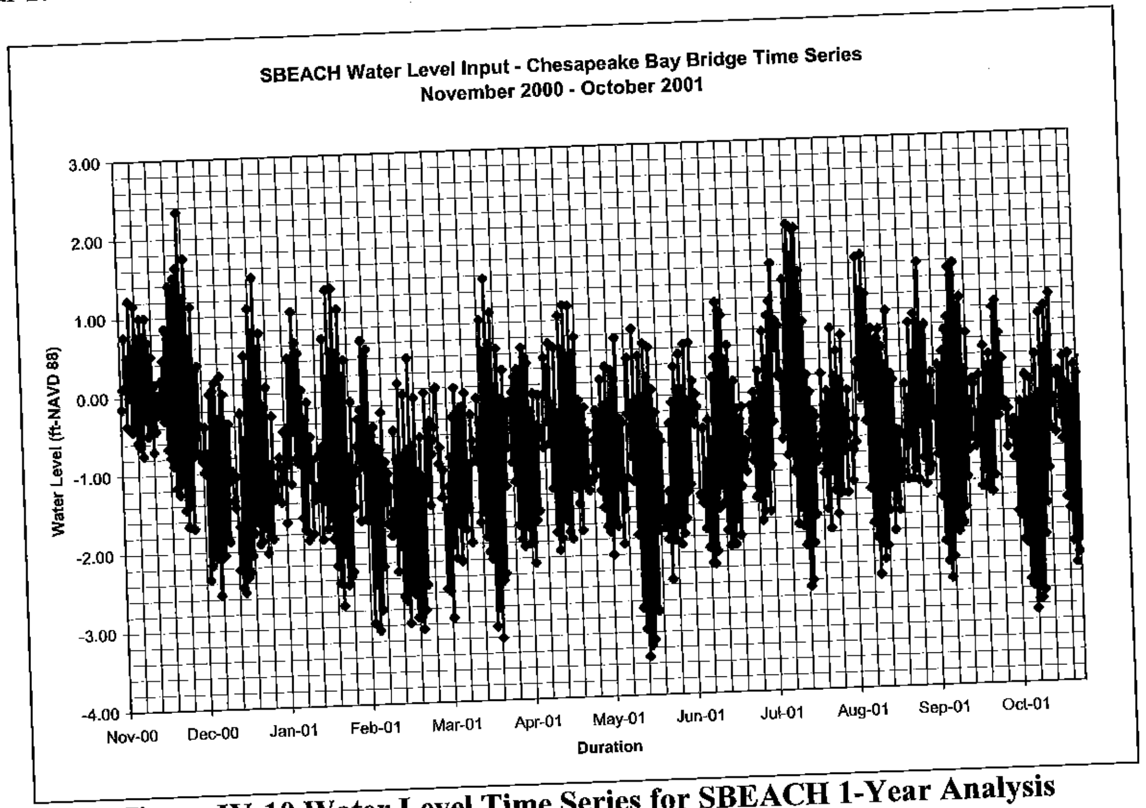


Figure IV-10 Water Level Time Series for SBEACH 1-Year Analysis

c) Sediment Data (1-Year Existing Conditions Model)

As described in **Section II-D** of this report, since the post-fill sediment data were not received until late April 2004, the pre-nourishment sediment characteristics were used in this analysis. Since the post-fill sediment grain size was expected to be coarser than the pre-nourished beach condition, the use of the pre-fill data was considered a conservative approach, since finer sediment is more likely transported offshore. Therefore, the effective grain size of 0.23 mm, used in the calibration model, was maintained in this existing conditions SBEACH model.

d) Model Calibration Coefficients (1-Year Existing Conditions Model)

The sediment transport parameters and corresponding parameters influencing sediment transport were defined by the model calibration analysis. Therefore, the existing conditions model was run using the following coefficients:

- Transport Rate Coefficient, K (m^4/N) = 2.5 x 10-7,
- Coefficient for Slope-Dependent Term, Eps (m2/s) = 0.0001,
- Transport Rate Decay Coefficient Multiplier = 0.5,
- Landward Surf Zone Depth = 1.6, and
- Avalanche Angle = 30°

e) Model Output (1-Year Existing Conditions Model)

In general the existing conditions, post-fill SBEACH model results showed that some erosion of the berm face (average 20 ft erosion setback at +3 ft NAVD 88) and consequential steepening of the beach slope can be expected to occur during the first year following construction. This cross-shore loss is based on the impact of typical wave and water level conditions for a one-year time period and the resulting profile is considered an equilibrium profile position.

During the SBEACH modeling, it was noticed that the profiles which crossed offshore breakwaters yielded inaccurate model results. As stated, SBEACH is designed to simulate storm induced erosion on the beach and/or berm. In this analysis, the simulated water levels and waves did not typically overtop the breakwaters. Furthermore, SBEACH does not allow for the breakwaters to transmit waves, but considers it an impermeable hardened structure. Later GENESIS modeling showed this was not the case. Therefore, in this modeling application, with the purpose of estimating the equilibrium profile, the results for the profiles crossing breakwaters were discarded.

Figure IV-11 shows an example of the SBEACH output profile following the 1-year simulation for Station 4+00, near the western edge of the project area. All of the remaining SBEACH output for the representative station profiles (excluding those crossing breakwaters) are shown in **Appendix H**.

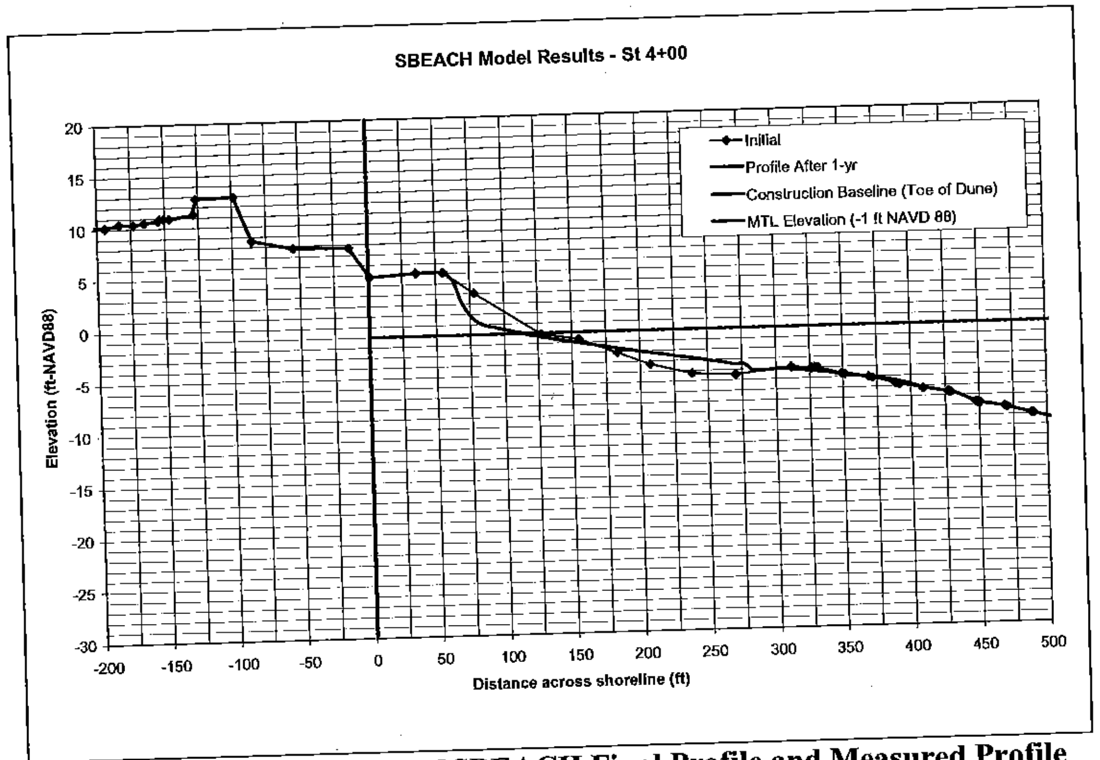


Figure IV-11 Comparison of SBEACH Final Profile and Measured Profile for Existing Conditions 1-Year Analysis Model

As stated previously, the final profiles in this SBEACH analysis were considered equilibrium profiles and were used to shift the initial shoreline position to represent an equilibrium position for the GENESIS long-term modeling. The mean tide level contour, estimated at -1 ft NAVD 88 served as the defined shoreline position for the existing conditions GENESIS modeling. Therefore, using the SBEACH model output, the erosion distance yielded for each profile at the -1 ft contour was determined. On average the profiles modeled (excluding those crossing breakwaters) eroded approximately 12 ft at the -1 ft contour, with maximum and minimum erosion distances of 11 ft and 34 ft, respectively. Using these erosion distances, the initial shoreline was shifted and interpolated between to create an equilibrium shoreline position. For those positions behind breakwaters, the -1 ft contour was assumed to be stable and the initial shoreline was not shifted. The resulting shorelines and their incorporation in the long-term modeling will be discussed in more detail in **Section V-C** of this report.

C. SBEACH MODEL – HURRICANE ISABEL ANALYSIS

To evaluate storm-induced impacts at the project site, an SBEACH model simulating the impact of Hurricane Isabel on the site following one year of typical wave and water level impacts was developed. This analysis was a stand alone application which was not incorporated into the overall analysis of expected design life of the beach fill. Since this study involved the simulation of long-term and average conditions, it was necessary to also evaluate the potential impacts of a storm event considered to be significant for the study area. Furthermore, the availability of pre- and post-storm measured beach surveys allowed for an improved analysis involving model calibration.

1. Model Calibration

Since pre- and post-Hurricane Isabel survey data was available for the study area, the SBEACH model was calibrated initially to determine the appropriate model coefficients. The model calibration involved selecting a couple of representative profiles to model and collecting the storm data, including wave, wind, and water level data for Hurricane Isabel.

a) Profile Data (Hurricane Isabel Calibration Model)

The profile data used in this modeling analysis included the June 2003 (pre-storm) and September 2003 (post-storm) beach and bathymetric surveys collected by Waterway Surveys & Engineering. For the purposes of the model calibration, an individual representative profile was selected. As shown in **Figure IV-12**, the selected profile was located at Station 9+00 of the beach fill project extent, at the western end of the study area to minimize the breakwater impacts. The June 2003 profile became the initial profile in the SBEACH model and the September 2003 was the final measured profile against which the SBEACH results were compared and calibrated. **Figure IV-13** shows the measured pre- and post-storm elevations for the selected profile.

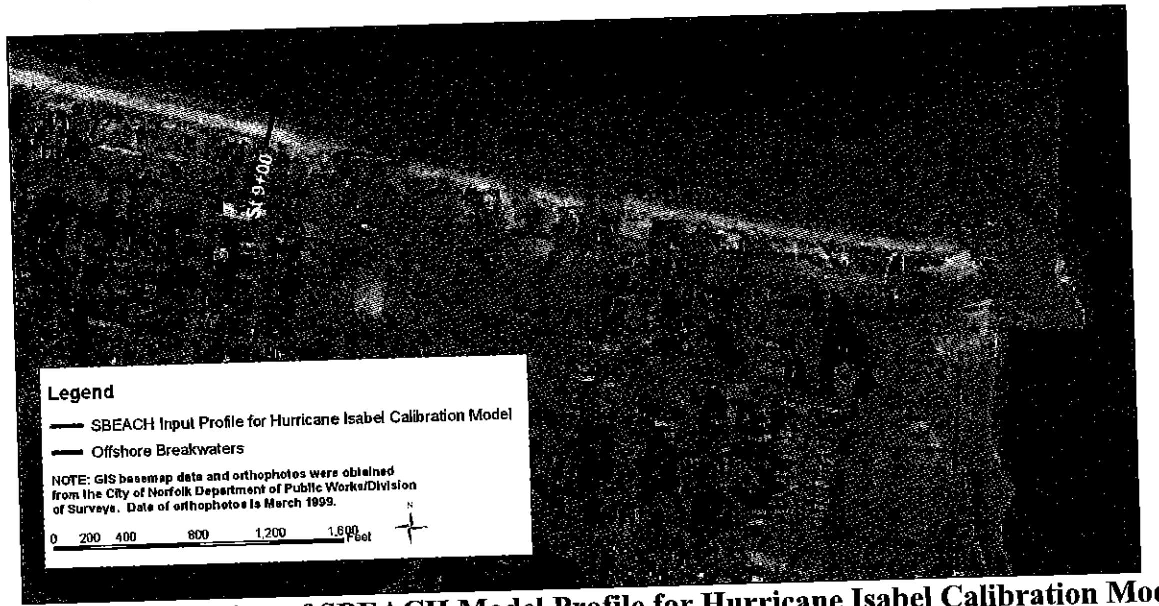


Figure IV-12 Location of SBEACH Model Profile for Hurricane Isabel Calibration Model

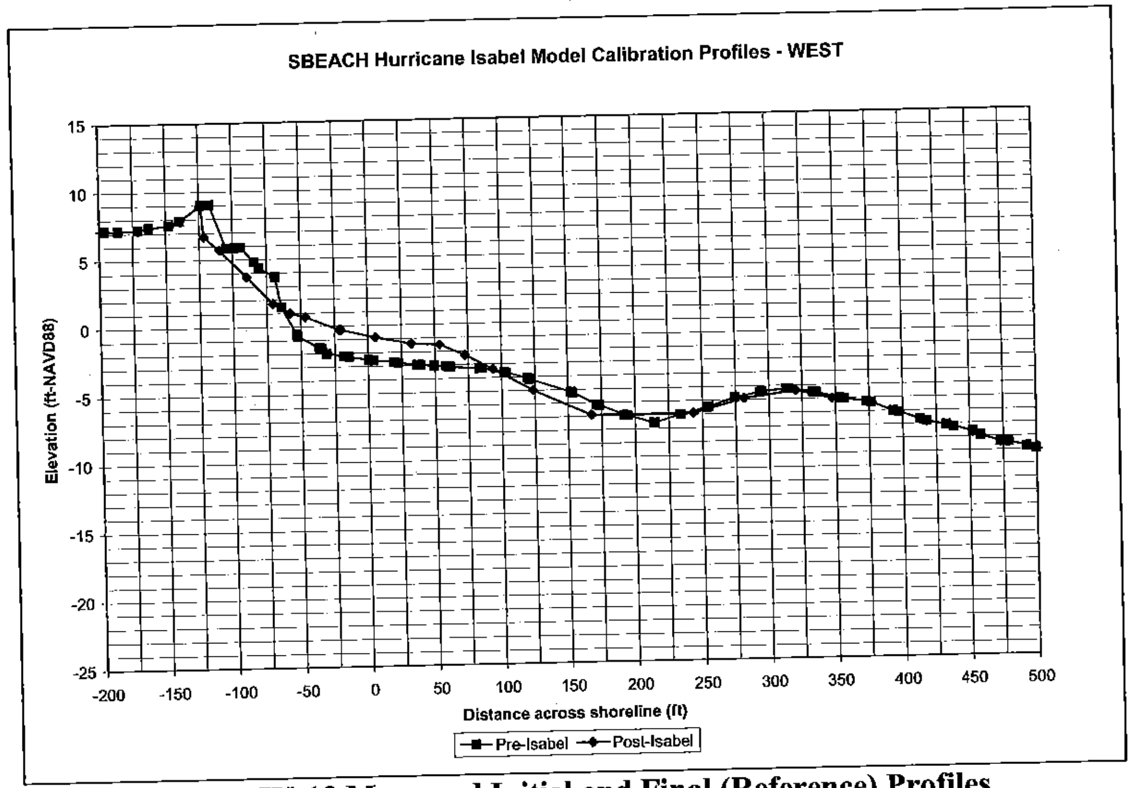


Figure IV-13 Measured Initial and Final (Reference) Profiles for SBEACH Hurricane Isabel Calibration Model

b) Storm Data (Hurricane Isabel Calibration Model)

The required storm data for the Hurricane Isabel SBEACH model included time series hydrographs of wave height, wave period, water level, and wind speed. These time series data sets were obtained from gage data collected near or offshore of the site. All of the applicable gages collecting measured wave, wind, and tide data, identified in the data collection portion of this study (**Figure II-1**) as well as any storm-specific data available for Hurricane Isabel were analyzed.

For obtaining the wave data time series, the closest available measured data were considered. VIMS deployed an Acoustic Doppler Current Profiler (ADCP) to measure storm related data (waves, tide, etc.) for Hurricane Isabel. This gage was located in 28 ft of water several hundred yards seaward of the VIMS campus site. While this gage provided consistent measured data for the storm event, its location was within a sheltered portion of the bay which likely had unique wave and tidal conditions. Therefore, it was not reasonable to assume that this data could be directly applied to the East Ocean View site.

Because the East Ocean View site was exposed to the Chesapeake Bay and the Atlantic Ocean, we elected to use the closest available offshore wave gage and transform the wave data, as needed to represent nearshore conditions. The closest measured wave data was at the NOAA C-MAN Chesapeake Light gage just offshore of the mouth of the bay (**Figure II-1**). This gage had measured wave heights and periods available for a majority of the storm duration, with intermittent missing measurements. These intermittent missing values were estimated through interpolation. The duration of the storm event for modeling purposes was defined by the duration of the wave height hydrograph, or the portion of time the waves exceeded typical values (approximately 5 ft offshore). The resulting storm duration was 6 days (144 hrs) extending from September 16 to September 22, 2003. **Figures IV-14** and **IV-15** show the measured offshore wave height and wave period hydrographs taken from the Chesapeake Light gage for Hurricane Isabel.

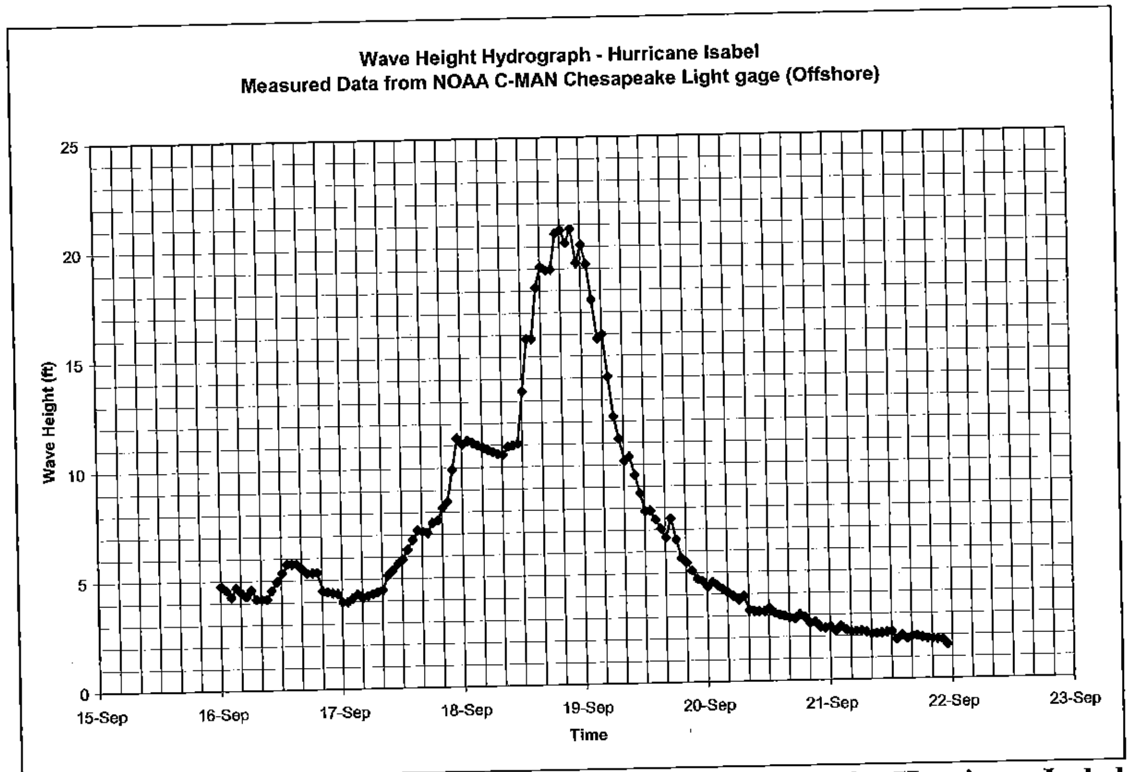


Figure IV-14 Measured Offshore Wave Height Hydrograph for Hurricane Isabel (Chesapeake Light gage)

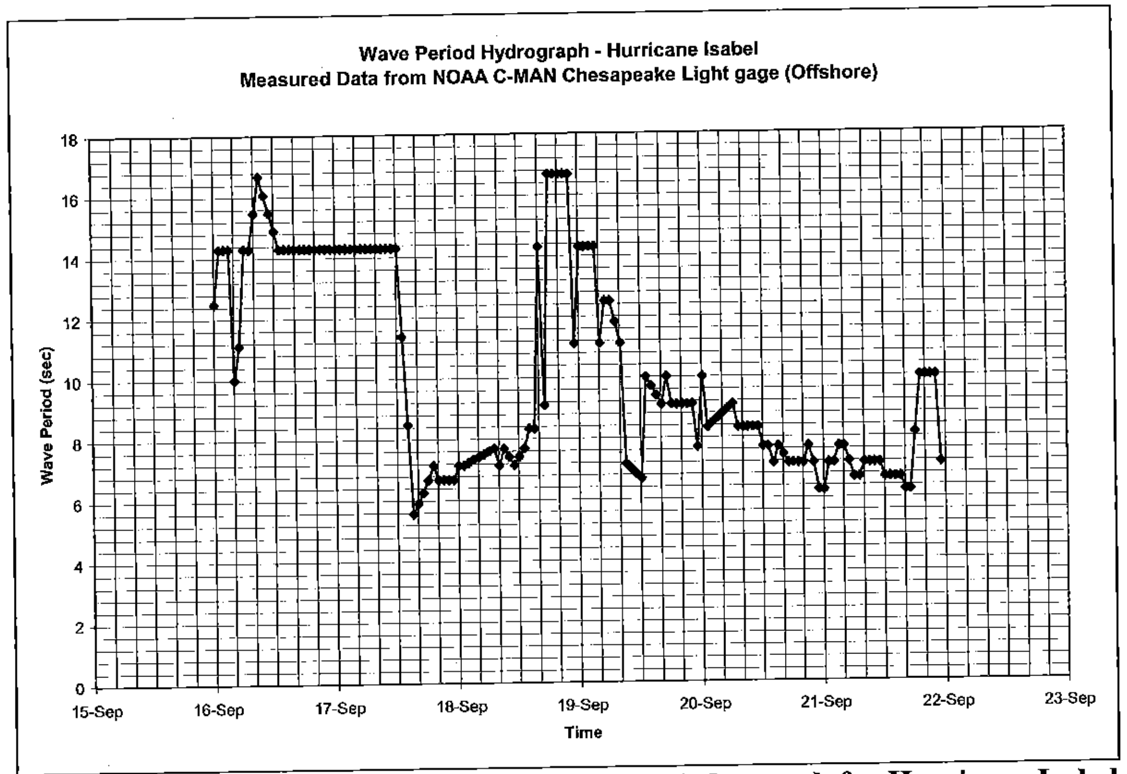


Figure IV-15 Measured Offshore Wave Period Hydrograph for Hurricane Isabel (Chesapeake Light gage)

As shown in **Figure IV-14**, the measured wave heights offshore of the site exceeded 20 ft at the peak. These wave heights represented extreme offshore conditions, and were again not indicative of the nearshore wave conditions experienced at the East Ocean View site. Therefore, a refraction coefficient was applied to reduce the wave heights to represent nearshore conditions. An average refraction coefficient of 0.2 was assumed based on the NSW model output (see **Section III-D**) used in transforming the swell wave data. After applying this factor to the entire wave height hydrograph, the resulting wave height time series to be used in the SBEACH model input was finalized. The resulting nearshore wave height hydrograph is shown in **Figure IV-16**.

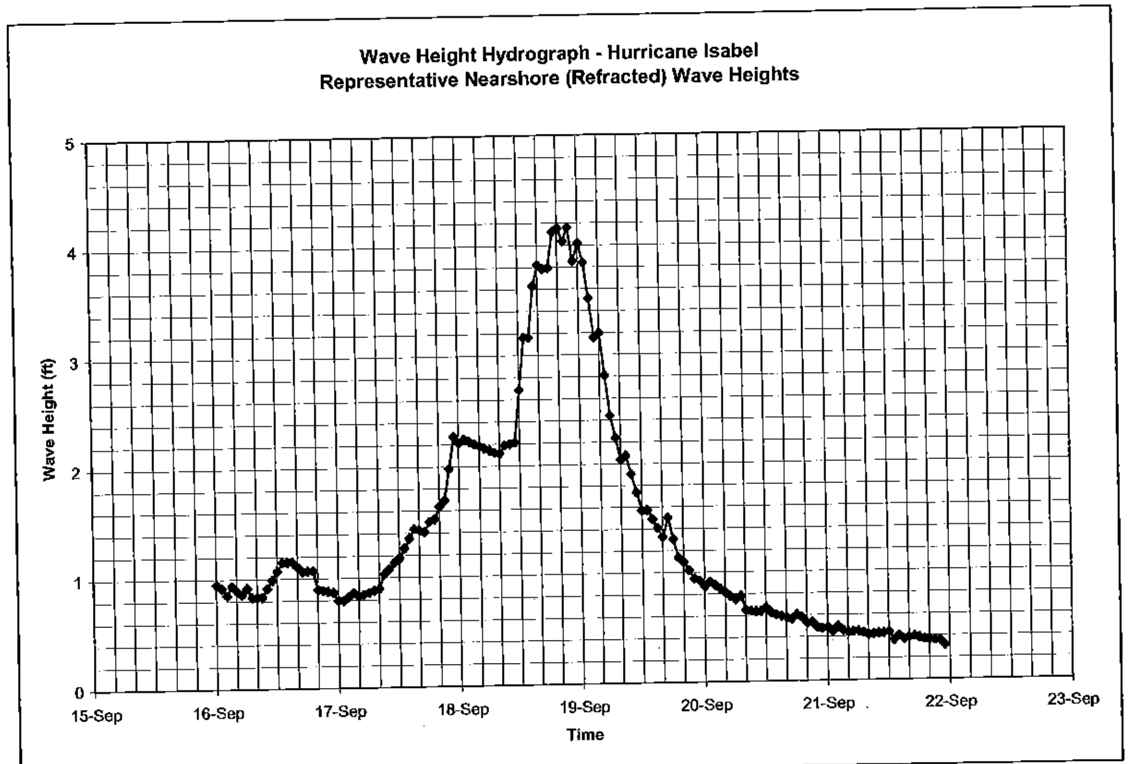


Figure IV-16 Refracted Nearshore Wave Height Hydrograph for Hurricane Isabel

As shown, the peak wave height reached approximately 4.2 ft at the nearshore location. This peak wave height was also consistent with the measured data collected offshore of the VIMS site, which recorded a peak wave height of approximately 5 ft for Hurricane Isabel. Therefore, this wave height time series was assumed valid for use in the SBEACH model. The wave period time series was not adjusted and was assumed to represent nearshore conditions (**Figure IV-15**).

The water level and wind data for Hurricane Isabel were obtained from the NOAA/NOS Chesapeake Bay Bridge Tunnel gage, as used for the other modeling analyses. These data time series were obtained for the duration established by the wave height hydrograph (144 hrs, Sept. 15 – 22, 2003). The water level data were adjusted to elevations above NAVD-88, as done previously. The resulting water level hydrograph and wind speed hydrograph are shown in **Figures IV-17** and **IV-18**.

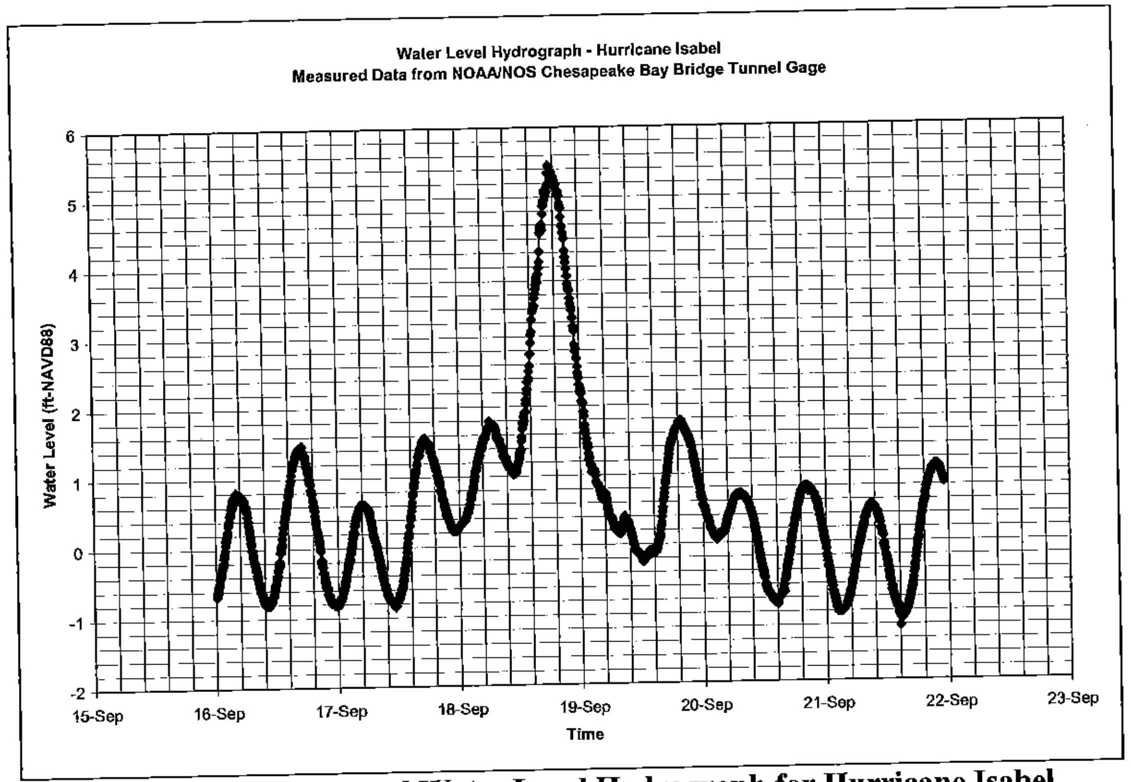


Figure IV-17 Measured Water Level Hydrograph for Hurricane Isabel (Chesapeake Bay Bridge Tunnel Gage)

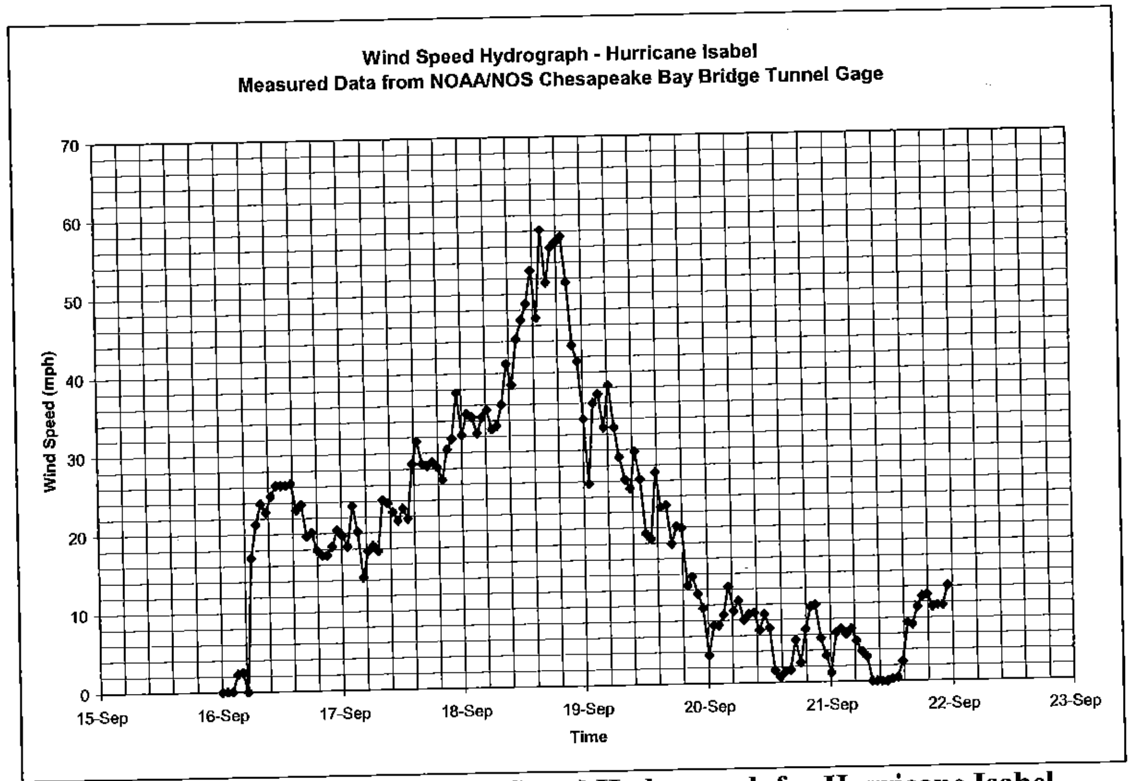


Figure IV-18 Measured Wind Speed Hydrograph for Hurricane Isabel (Chesapeake Bay Bridge Tunnel Gage)

c) Sediment Data (Hurricane Isabel Calibration Model)

Again, since Hurricane Isabel occurred before the nourishment project, the pre-project effective grain size of 0.23 mm was used in the calibration model.

d) Model Calibration Coefficients (Hurricane Isabel Calibration Model)

Since the actual beach and sediment conditions should be consistent regardless of the type of storm being simulated, we began with the model calibration coefficients established during the previous modeling. Therefore, the Hurricane Isabel SBEACH calibration model was run using the following coefficients:

- Transport Rate Coefficient, K (m^4/N) = 2.5 x 10-7,
- Coefficient for Slope-Dependent Term, Eps (m2/s) = 0.0001,
- Transport Rate Decay Coefficient Multiplier = 0.5,
- Landward Surf Zone Depth = 1.6, and
- Avalanche Angle = 30°

Additionally, the SBEACH model was run using coefficients established in a similar study by M&N which involved the use of SBEACH to model the impacts of hurricanes and noreasters along Ocracoke Island, NC. These coefficients were as follows:

- Transport Rate Coefficient, K (m^4/N) = 1.75 x 10-7,
- Coefficient for Slope-Dependent Term, Eps (m2/s) = 0.002,
- Transport Rate Decay Coefficient Multiplier = 0.5,
- Landward Surf Zone Depth = 1.0, and
- Avalanche Angle = 30°

e) Model Output (Hurricane Isabel Calibration Model)

The SBEACH model output showed that the model results did not differ significantly using the two different sets of model coefficients. Therefore, to maintain consistency in the SBEACH model applications of the study area, the coefficients applied in the existing conditions model were used. A comparison of the final model profile and the measured final profile is shown in **Figures IV-19**.

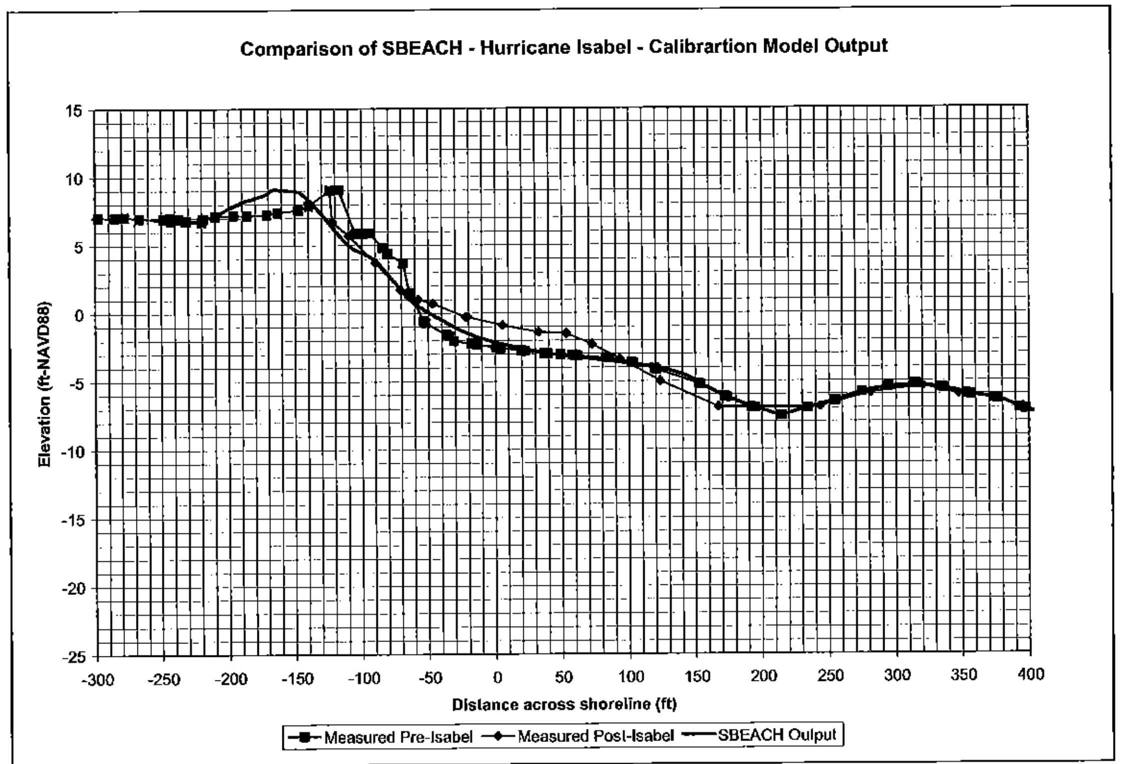


Figure IV-19 Comparison of SBEACH Final Profile and Measured Profile for Hurricane Isabel Calibration Model

As shown, the final SBEACH profile matches well with the measured post-storm profile, particularly along the berm face, where a large portion of erosion can be expected to occur. There is some difference in the measured and final profiles between elevations 0 ft and -3 ft, where the measured post-storm profile shows accretion. This accretion is likely a result of the shifting of loose sand offshore and resulting berm formation which typically occurs over a period of time following significant storm events. Therefore, the SBEACH results can not be expected to show this formation.

2. *Hurricane Isabel SBEACH Model*

As stated, an SBEACH model simulating the impact of Hurricane Isabel on the site following one year of typical wave and water level impacts was developed using the calibration model coefficients defined in the previous analysis and in the 1-year analysis. The goal of this modeling application was to simulate how a significant storm event, similar to Hurricane Isabel would impact the nourished site after the site achieved an equilibrium condition, following 1-year of typical wave and water level action.

a) <u>Profile Data (Hurricane Isabel Model)</u>

The initial profile data used in the SBEACH model of Hurricane Isabel consisted of the SBEACH output from the 1-year average conditions simulation. Therefore, the input profiles represented the same cross-sections defined in **Table IV-1** and **Figure IV-6**.

b) Storm Data (Hurricane Isabel Model)

The storm data used in the Hurricane Isabel model were the same wave height, wave period, water level, and wind speed hydrographs developed for the calibration model as described in the previous section.

c) Sediment Data (Hurricane Isabel Model)

Again, given the unavailability of post-nourishment sediment data until after April 2004, the effective sediment grain size used in this analysis was the characteristic pre-nourishment average d50 of 0.23 mm.

d) Model Calibration Coefficients (Hurricane Isabel Model)

The sediment transport parameters and corresponding parameters influencing sediment transport were defined by the model calibration analysis and are as follows:

- Transport Rate Coefficient, K (m^4/N) = 2.5 x 10-7,
- Coefficient for Slope-Dependent Term, Eps (m2/s) = 0.0001,
- Transport Rate Decay Coefficient Multiplier = 0.5,
- Landward Surf Zone Depth = 1.6, and
- Avalanche Angle = 30°

e) Model Output (Hurricane Isabel Model)

The results of the SBEACH model run of Hurricane Isabel on the post-fill profiles after one year of average wave and water level impact, did not show significant loss from this storm event. There was some additional loss of the berm face (average erosion setback of 35 ft at elevation +3.0 ft-NAVD 88), however the dune system was maintained and did not experience significant loss from the storm impact. Nonetheless, as normal erosional processes occur over time, there would be a time when a storm event such as Hurricane Isabel would impact the dune system and possibly breach it. A typical results graph for Station 4+00 is shown on **Figure IV-20**. On this graph, the initial profile (post-fill), resulting profile after one year of typical impacts (also, the initial profile in the Hurricane Isabel analysis) and the post 1-year profile after the impact of Hurricane Isabel are all shown. The remaining results graphs for the other profiles which were included in the model are shown in **Appendix I**.

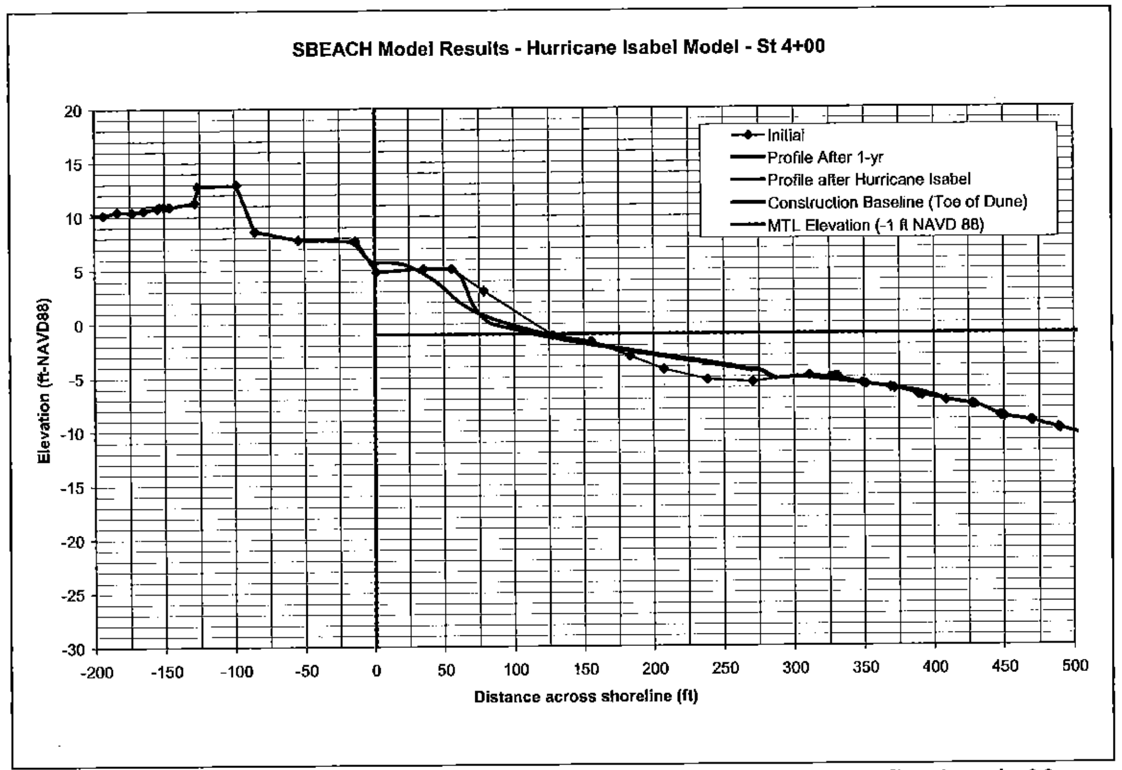


Figure IV-20 SBEACH Hurricane Isabel Model Results for Station 4+00

V. GENESIS MODEL

GENESIS is the Generalized Model for Simulating Shoreline Change. This model is designed to simulate long-term shoreline change based on spatial and temporal differences in longshore sediment transport induced primarily by wave action. The GENESIS modeling system allows for a number of user-specified inputs including wave inputs, initial shoreline positions, coastal structures and their characteristics, and beach fills; all of which aid in the calculation of sediment transport and shoreline change. This model was developed at the U.S. Army Corps of Engineers (USACE) Waterways Experiment Station (WES) Coastal & Hydraulics Laboratory (CHL). For a more detailed description of the GENESIS model, the reader is referred to a User's Manual and Technical Reference published on the model (Hanson and Krauss, 1989, Gravens et al, 1991).

The most recent version of GENESIS, released in 2002, operates under the Coastal Engineering Design and Analysis System (CEDAS), a suite of tools developed by Veri-Tech, based on various numerical models and codes developed at CHL. The CEDAS suite also includes BMAP (Beach Morphology Analysis Package) and SBEACH (Storm Induced BEAch CHange), tools which were utilized in this study. GENESIS operates under NEMOS, which is designed to ease in the preparation of data inputs, analysis, and manipulation for a number of related coastal models.

The GENESIS model has potential for many applications in the coastal environment, including evaluation of longshore sediment transport, analysis of beach fill performance, or the analysis of the impact of coastal structures on shoreline change.

The main inputs to the GENESIS model include:

- Shoreline Position Data – one-dimensional description of the shoreline position relative to a straight baseline position,
- Wave Data – long-term time dependent description of wave heights, periods, and directions applicable to the study area,
- Coastal Structures – position and characteristics of coastal structures (breakwaters, groins, jetties, or seawalls) acting along the study area,
- Sediment Data – characteristic effective grain size for the study area, and
- Boundary Conditions – seaward boundary conditions for the input wave data and lateral boundary conditions for the shoreline (left and right).

A. SCOPE OF GENESIS MODELING

The GENESIS model served as the basis for determining the expected design life of the beach fill project completed in December 2003. The scope of the GENESIS modeling task for the East Ocean View site involved evaluating the long-term change in shoreline position based on a long-term period of wave action (1991-2003). **The expected design life of the beach fill project was defined by calculating the time at which the post-fill shoreline position reached the pre-project, pre-Isabel shoreline position.** The post-project shoreline position used as the initial shoreline in the GENESIS model was shifted to represent the equilibrium shoreline position using the results of the 1-year SBEACH analysis, discussed in **Section III-B**.

To establish the appropriate model parameters, the GENESIS model was calibrated for a 1995-2002 time period using historical profile data and coinciding wave and tide data. GENESIS is calibrated by adjusting the longshore sand transport coefficients (K_1 and K_2). Additionally, the model may be calibrated by adjusting the characteristic transmissivity or permeability of offshore breakwaters, groins or jetties.

Once the GENESIS model was calibrated, a model based on the post-fill equilibrium shoreline (post 1-year shoreline change) was generated using the established calibration coefficients. This model included a 20-year simulation period (2003 to 2023) utilizing the nearshore wave data transformed from the Duck site. The results of this long-term model were used to determine at what point in the future the shoreline position reached the pre-nourishment shoreline position. This time period defined the estimated design life of the beach fill project.

B. GENESIS MODEL CALIBRATION

Calibration of the GENESIS model required the availability of historical shoreline positions and a reasonably long wave time series to simulate long-term shoreline change. The selection of the calibration time period was dependent on the nearshore wave time series which spanned 1991-2003. Furthermore, the calibration time period had to be selected such that the structural conditions on the beach were consistent. In other words, the model operates on the basis that the coastal structures (e.g. offshore breakwaters) are either absent or present and not time-dependent.

For the GENESIS modeling application involved in this study, effective model calibration required simulating both a long time period and a time period during which the offshore breakwaters were in place. An overall effective model calibration involved determining the appropriate breakwater transmissivity coefficients to apply in the post-fill model. However, since the breakwaters were built in 2000-2001, this model length would only span a maximum of 3 years. Based on the availability of shoreline positions, a modeling strategy which implemented both a reasonably long time series simulation and a period during which the breakwaters were in place was developed. Two GENESIS models were implemented: 1) an initial model spanning from 1995 to 1999 to calibrate the longshore transport coefficients and 2) a second model spanning 1999 to 2002 to calibrate the breakwater transmissivity coefficients. The second model from 1999 to 2002 utilized the established longshore transport coefficients and the resulting model shoreline from the previous time period simulation. The model development details for both models will be discussed, and results presented for the overall 1995-2002 calibration time period.

1. Shoreline Position Data (Calibration Model)

For shoreline input, the GENESIS model requires the shoreline be specified in a station-offset formulation whereby the station represents a position along a landward baseline and the offset is the perpendicular distance from this baseline to the shoreline. For the East Ocean View model, a GENESIS baseline was established which extended approximately along Ocean View Avenue, beginning near 17th Bay Street and ending at the Little Creek Inlet jetty. A transect was placed every 20 feet perpendicular to this line to estimate the shoreline position. **Figure V-1** shows the extent of the GENESIS model and the shoreline transects.

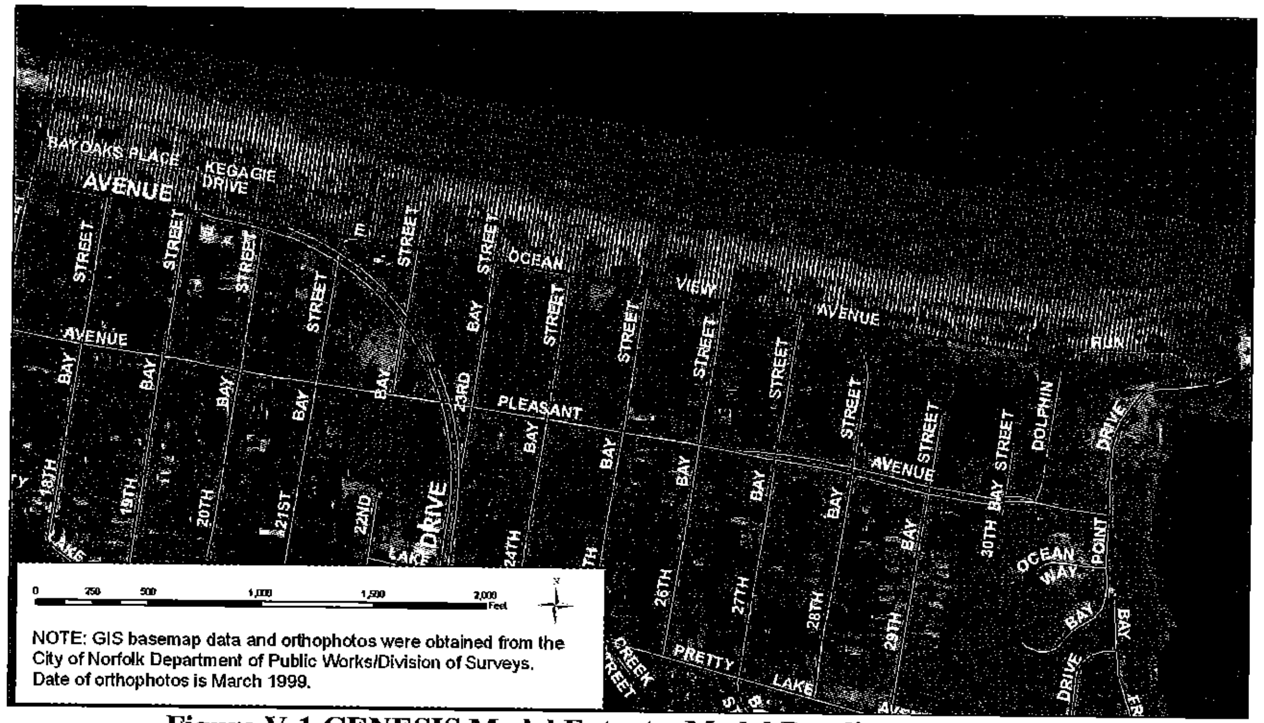


Figure V-1 GENESIS Model Extent – Model Baseline and Transects

For the calibration models, the digitized shoreline positions dating October 1995, October 1999, and June 2002 were used (see **Section II-C**). As discussed, these shoreline positions were modified from Dr. Basco's analysis, because the initial shoreline positions were too coarse and did not capture significant features which would be influential in the long-term modeling. The modified shorelines were developed by M&N by scanning the VIMS aerial photography, georectifying the images to previous photos of the study area, and digitizing the shoreline position as the approximate wet-dry line along the beach. **Figure V-2** shows the resulting digitized shorelines overlain on the 2002 scanned and georectified aerial photographs. The approximate positions of the offshore breakwaters are also indicated on this figure, although they were not officially in place until 2000-2001.

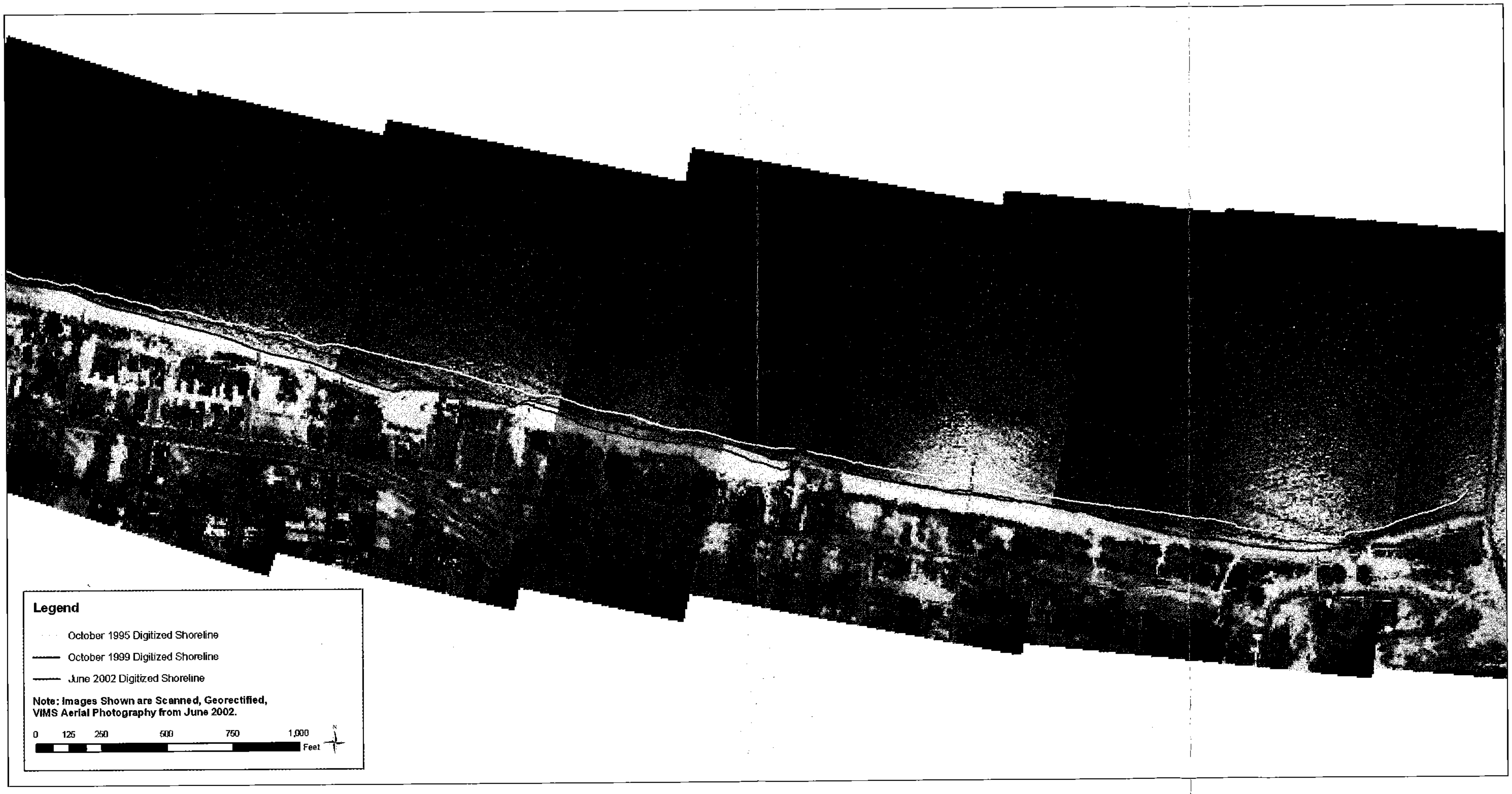


Figure V-2 Digitized Calibration Shorelines (1995, 1999, and 2002) for GENESIS Model

As shown in **Figure V-2**, between 1995 to 1999 the shoreline experienced overall erosion within the study area extent. From 1999 to 2002, the time period during which the offshore breakwaters were built, the shoreline was generally stable or accreting behind the breakwater positions and eroding outside of the breakwater field. These trends exhibit the significance of the offshore breakwaters and the need for an accurate calibration effort.

To obtain the GENESIS model inputs, the distances from the model baseline to each shoreline were measured at 20 ft intervals along the model baseline. For the first model, the initial and reference shorelines were the 1995 and 1999 positions, respectively. For the second model, the initial shoreline was not yet determined, as it was the output of the first model. The reference shoreline for the second model was the 2002 shoreline position.

GENESIS also requires the user to specify the depth of closure and an average berm elevation for the study area. For this model, the depth of closure was defined as 7 ft and the average berm elevation was set to +3 ft NAVD 88 for the overall time period of 1995-2002. These values were determined based on observations of the measured survey data during the calibration time period.

2. *Wave Data (Calibration Model)*

GENESIS allows the user to enter up to two separate wave components in the model. The model uses the offshore depth of the measured wave data (-20 ft NAVD 88), defined for each wave component to refract and shoal the waves towards the model shoreline. The required wave data inputs include time series of significant wave height, peak wave period, and wave direction. The nearshore wave data swell and sea components were used to develop the GENESIS model wave inputs. The wave data components were extracted for both model time periods (Oct 1995 to Oct 1999 and Oct 1999 to June 2002), coinciding with the measured shoreline positions. Each wave data time series had a 6-hour time step, which was a result of the controlling 6-hour measurements obtained at Duck.

Along with the wave data inputs, the model allowed for the user to enter water level data. The water level data was extracted from the Chesapeake Bay Bridge Tunnel gage data for each of the specified model time periods. This data was added to both the sea and swell wave components.

3. *Coastal Structures Data (Calibration Model)*

The GENESIS model can simulate a number of coastal structures and engineering activities including:

- Non-diffracting groins or jetties
- Diffracting groins or jetties
- Seawalls
- Detached Breakwaters
- Beach fills
- Sand bypassing

The coastal structures are incorporated in the GENESIS model by a station-offset formulation, similar to the shoreline position. Each structure is modeled uniquely with respect to longshore transport and shoreline change. In general structures exert two direct effects on the shoreline change modeling:

1. Long structures, extending into the surf zone block a portion or all of the longshore transport from their updrift sides and may reduce the transport of sand towards the downdrift side. This effect may be induced by a groin or jetty.

2. Structures which have seaward ends extending well beyond the surf zone induce wave diffraction which causes the local wave height and direction to change.

For the two sequential calibration models, a number of coastal structures were implemented in the model. Maps and descriptions of these coastal structures and their relation to the model are shown for the two calibration time periods in **Figures V-3** and **V-4**.

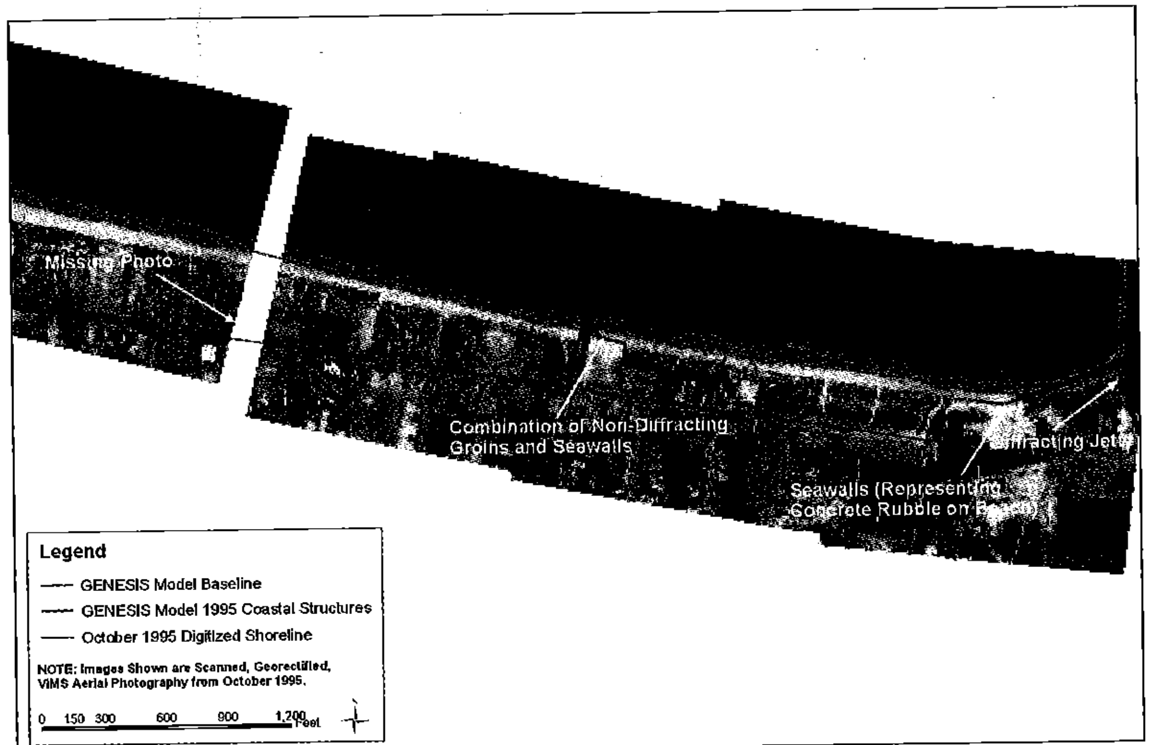


Figure V-3 Coastal Structures Included in 1995-1999 GENESIS Calibration Model

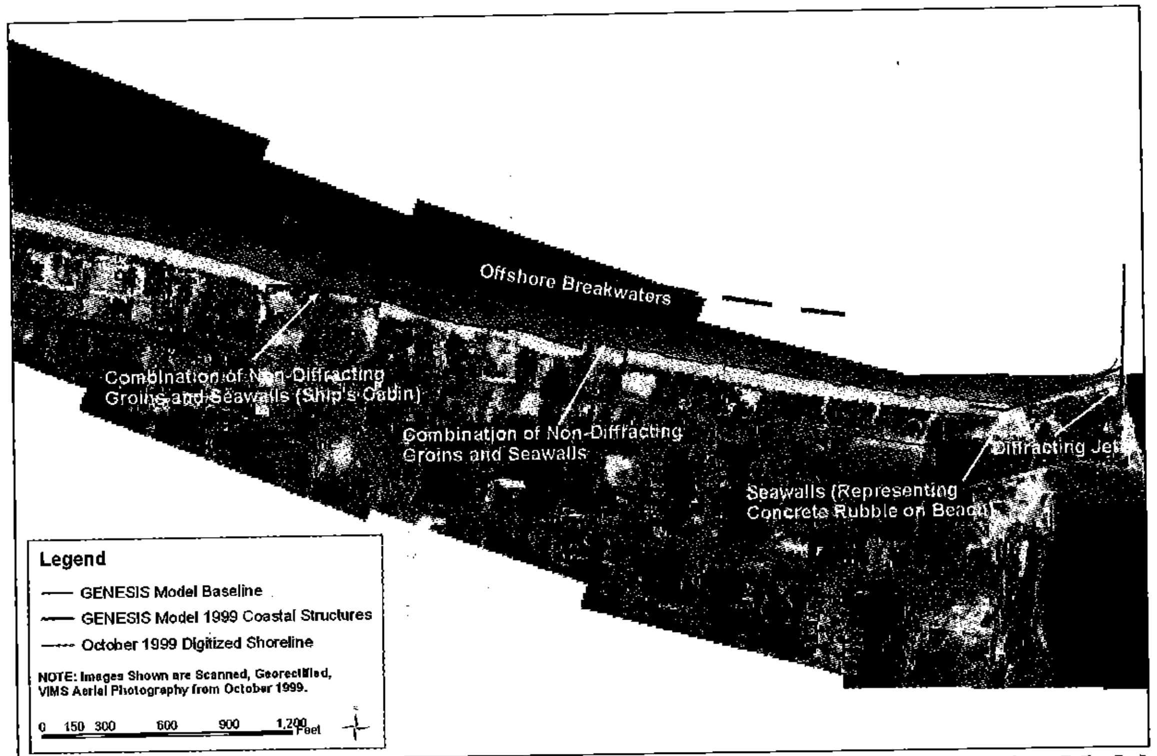


Figure V-4 Coastal Structures Included in 1999-2002 GENESIS Calibration Model

For groins, jetties, or breakwaters, the user must specify a permeability (groins, jetties) or transmissivity (breakwaters) coefficient which indicates the amount of wave energy for breakwaters which is passed through or over the structure. Seawalls are considered nonerodible, impermeable structures, which deter any shoreline movement in the landward direction. The Little Creek Inlet jetty was defined as impermeable (permeability coefficient = 0), based on general field observations of the structure and nearby shoreline characteristics. All non-diffracting groins used in the model were modeled as impermeable (0.0) to slightly (0.1) permeable. These coefficients were applied using observations of the shoreline positioning updrift and downdrift of these structures and based on the model calibration. Finally, the offshore breakwaters incorporated in the 1999-2002 calibration model were determined to be highly permeable and easily overtopped, yielding a transmissivity coefficient of 0.8 (80% wave transmission through and over the structure). Again, this value seemed reasonable given the low elevation of these structures and the large rubble mound formation which may allow for significant transmission of wave energy.

4. *Sediment Data (Calibration Model)*

GENESIS requires the user to enter a characteristic effective grain size (d50) for the model area. For the calibration models, an effective grain size of 0.23 was used, based on the pre-nourishment sediment data analysis of the study area.

5. Boundary Conditions (Calibration Model)

The required boundary condition inputs for the GENESIS model include the seaward wave data boundary conditions and the lateral boundary conditions at the left (west) and right (east) ends of the shoreline.

a) Seaward Boundary Conditions

Under the seaward boundary conditions the user must specify the following parameters:

- **Number of wave components applied** – a value of 1 or 2 is entered depending on how many wave time series files are being implemented
- **No of cells in the offshore contour smoothing window** - an indication of how the offshore contour moves relative to the shoreline in the internal wave model; used to prevent unrealistic wave transformation that may occur if the shoreline changes relatively abruptly (e.g. at a groin); a lower value indicates the offshore contour follows the shoreline position whereas a higher value "smooths" the contour making it straighter than the shoreline.
- **Input wave adjustments** – User may define a wave height amplification or direction factor which is applied to the input wave time series to adjust the wave height or direction; user can also apply a wave angle offset which is used to adjust the wave directions (a positive value is added to the wave directions and a negative value is subtracted).

As stated, the number of wave components applied was set to 2, since the model included both the sea and swell time series data. After several trials, the smoothing factor was set at 35 (maximum allowable = 50), based on the effect that this parameter was found to have on the model shoreline.

The only input wave adjustment applied was a wave angle offset. During initial model runs, it was determined that the concentration of waves arriving from the Northeast between 0° to 60° was high in both the sea and swell data time series (see **Figures III-10** and **III-12**). The internal wave model applied in GENESIS was causing the model to create a deep scour in the shoreline just west of the seawalls near the Little Creek jetty and transport this material in the westerly direction. Therefore, there was excessive erosion shown near the jetty, but very little erosion shown towards the west. Given the availability of the measured final shoreline positions, it could be observed that this behavior was not indicative of the actual long-term erosion and sediment transport occurring at the site. Upon adjusting several model parameters, it was determined that shifting the directions of the waves, by applying an angle offset, eliminated this scouring and resulted in a more uniform erosional pattern that matched the measured shoreline at the eastern end. However, the western portion of the shoreline remained stable despite the change in wave direction.

Since the measured shoreline positions indicated a more consistent, uniform erosional pattern (see **Figure V-2**), the GENESIS model was broken into two submodels representing the east and west sections. The separation of the east and west sections of the study area prevented the unrealistic transport of material from the east to west and allowed for each area to be treated uniquely with respect to wave impacts. As shown in **Figure V-5**, the model was divided at a position approximately halfway along the study area, near the concrete structure seaward of 25th Bay Street. This position was chosen because it was considered a pinned position where the shoreline was consistent over time. The division of the study area resulted in east and west models which were run independently but yielded results which could be combined at the pinned division point. The wave directions were shifted by +18° for the east model. According to the GENESIS standard for wave direction input, this offset shifted the wave directions in a counterclockwise manner or more towards the northwest. No angle offset was applied on the west model.

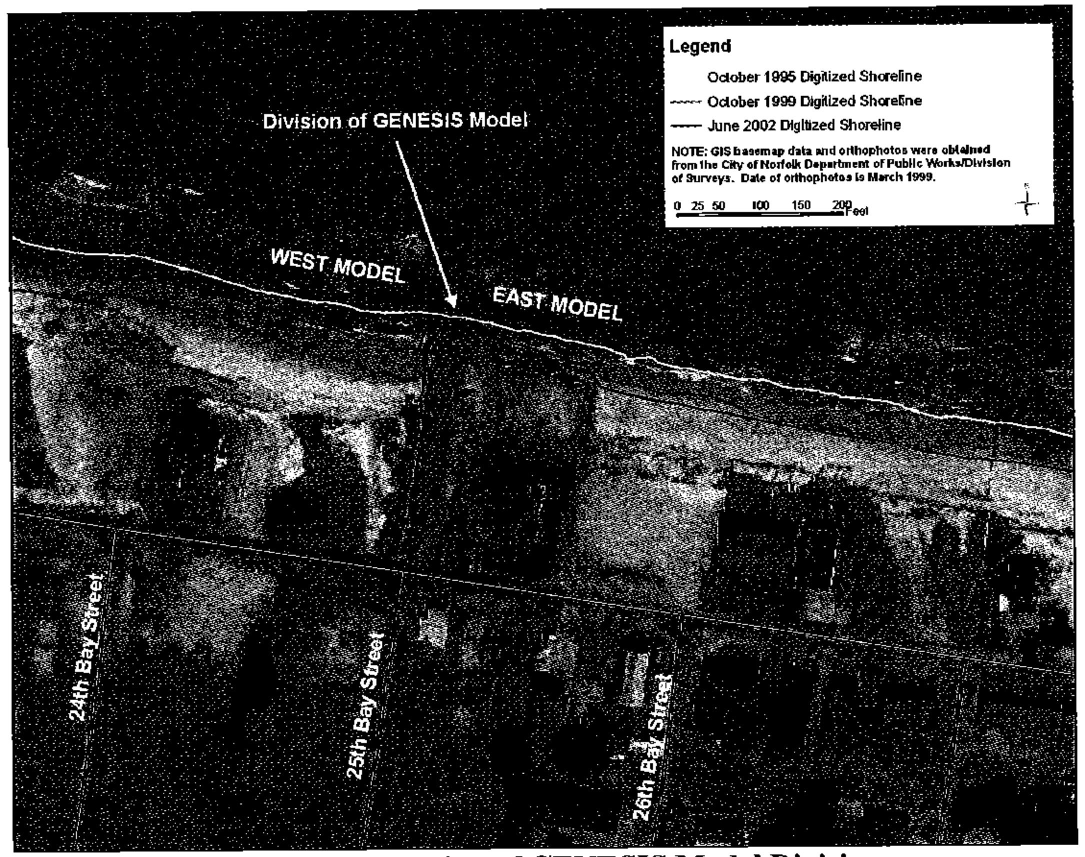


Figure V-5 Location of GENESIS Model Division

b) Lateral Boundary Conditions

The lateral boundary conditions specified by the user proved to have a significant impact on the GENESIS model results and the accuracy of the calibration results. GENESIS allows for three different boundary condition types: 1) a "pinned" boundary where the shoreline is considered stable over time, 2) a "gated" boundary condition where there is some gain or loss of sediment at the boundary, or 3) a "moving" shoreline where the user specifies an erosion or accretion rate for the shoreline.

Unique boundary conditions were defined for the different model simulations (1995-1999 and 1999-2002) depending on the differences in the measured shoreline positions observed at the boundaries. First, at the left or west boundary of the west model (near 17th Bay Street) either a moving or pinned shoreline was defined based on the observed change between the initial and final shoreline positions. At the right or east boundary of the east model, a gated boundary condition was implemented since the measured shoreline positions just west of the jetty showed some change over time. This is a typical condition to apply for a boundary where a diffracting jetty is in place. Finally at the division point for the east and west models, the boundary conditions were selected as pinned or moving based on the observed shoreline change downdrift and updrift of the structure. The presence of the breakwaters in the 1999-2002 calibration also forced a pinned boundary at this location, since the model division point was located behind a breakwater. As shown in **Figure V-5** above, even though the face of the structure itself maintains a stable shoreline position, the adjacent shorelines indicate some erosion, particularly on the updrift side for the 1995-1999 time period. Therefore, both moving and pinned boundaries were used to achieve the desired calibration model results. **Figure V-6** shows a summary of the general boundary conditions implemented for the calibration model simulations.

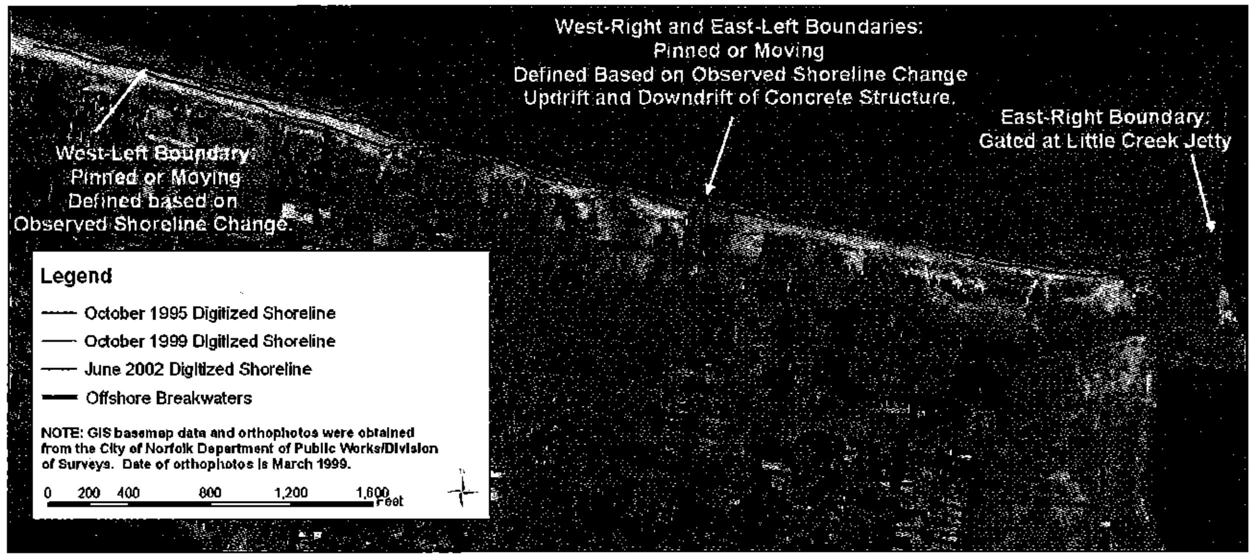


Figure V-6 Summary of GENESIS Calibration Model Boundary Conditions

6. Model Calibration Coefficients (Calibration Model)

As stated, the GENESIS model is calibrated by adjusting the K-factors which characterize the longshore sediment transport in the model. Also, the model may be calibrated by adjusting the permeability and/or transmissivity of the shoreline structures included in the model.
The first GENESIS model spanning 1995 to 1999 was used to develop the appropriate K-factors based on calibration with the measured 1999 shoreline position. An initial model was run using typical K_1 and K_2 values of 0.5 and 0.25 respectively. The resulting 1999 model shoreline was compared with the measured 1999 shoreline and the K factors were adjusted to achieve the closest match in the model results and the measured shoreline position. Through this procedure, it was determined that reducing the K values resulted in less sediment being transported onto the model study area, and therefore yielded a more accurate change in the shoreline over time. The final calibration coefficient values which yielded the most accurate shoreline were $K_1 = 0.25$ and $K_2 = 0.18$.

Having established the appropriate K-factors to characterize the longshore sediment transport, the second calibration model simulating 1999-2002 was implemented to determine the adequate breakwater transmissivity. The initial shoreline position used in this model was the final calculated GENESIS shoreline (not the measured 1999 shoreline) from the previous calibration of 1995-1999. Based on field observations, the initial breakwater transmissivity was believed to be high given that the breakwater crests were at a lower elevation and the rubble mound structures allowed for transmission through the structure. Again, a series of models were run with adjusted transmissivity coefficients until a reasonable calibration was achieved between the model and measured shoreline positions. As stated, a transmissivity value of 0.8 was found to yield the most accurate results.

7. *Model Output (Calibration Model)*

Figure V-8 shows the results of the overall calibration analysis, including the initial (1995) shoreline position, the final 2002 model shoreline and the final measured 2002 shoreline.

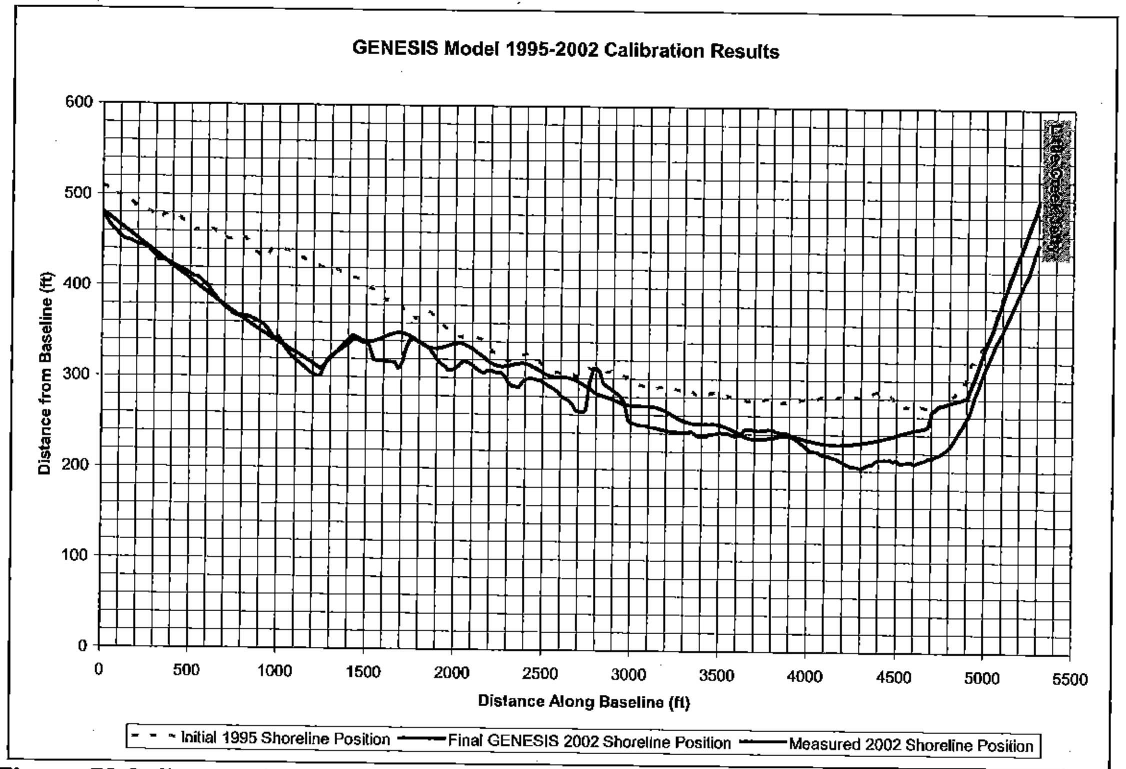


Figure V-8 Comparison of Overall GENESIS 1995-2002 Model Calibration Output

As shown, the model output matches reasonably well with the measured 2002 shoreline. Based on these results, the calibration parameters including the K-factors and the offshore breakwater transmissivity were considered adequate for use in the existing conditions model.

C. EXISTING CONDITIONS (POST-FILL) GENESIS MODEL

Having established the appropriate model coefficients, an existing conditions (post-fill) GENESIS model was developed to estimate the expected design life of the beach nourishment project. The definition of expected design life was determined by the time period over which the initial shoreline position reached the pre-nourishment shoreline position. The model simulation time period was 20 years beginning in December 2003.

1. Shoreline Position Data (Existing Conditions Model)

The basis for the initial shoreline in this model was the post-fill shoreline extracted at the approximate MTL elevation (-1 ft NAVD 88) and shifted to account for the initial 1-year losses. Again, the initial 1-year losses were determined in the existing conditions SBEACH model, as discussed in **Section IV-B**. The pre-nourishment shoreline was taken from the June 2003 survey data at the MTL elevation and served as the reference shoreline in this analysis. The same baseline and model transects as used in the calibration analysis, were applied here. The GENESIS model input was determined by measuring the distance from the baseline to the shoreline at each transect. **Figure V-10** shows the resulting initial shoreline and the reference pre-nourishment shoreline overlain on the City of Norfolk orthophotos (dating March 1999).

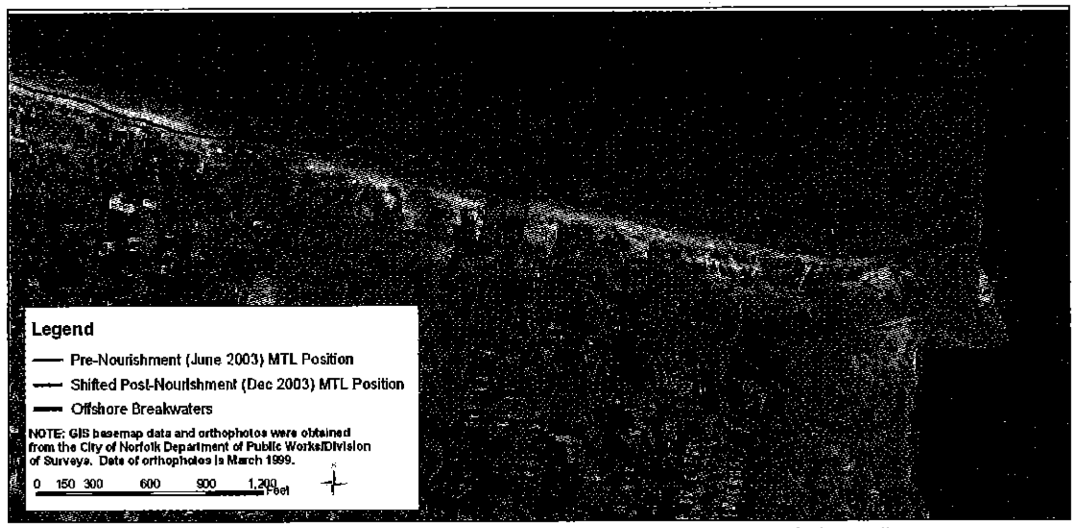


Figure V-10 Initial (Dec 2003, Shifted Post-Fill) and Pre-Nourishment (June 2003) MTL Shoreline Positions

As stated, GENESIS also requires the user to specify the depth of closure and an average berm elevation for the study area. For this model, the depth of closure remained at 7 ft and the average berm elevation was increased to +5 ft NAVD 88 for the post-fill conditions.

2. Wave Data (Existing Conditions Model)

The wave data used in the existing conditions, post-fill analysis consisted of the long sea and swell wave time series dating January 1991- December 2003. Therefore, the historical transformed wave data served as a representative time series of typical wave conditions that can be expected in the future. As done for the calibration model, time series of significant wave height, wave period, and wave direction were entered for the sea and swell components. Additionally, the measured water level data coinciding with the measured wave data (1991-2003) were implemented with the wave data components. Both the wave and water level data time series had a 6-hour time step, which was a result of the 6-hour measurements obtained at Duck. As noted, the GENESIS simulation was for a 20-year time period. In order to provide a 20-yr wave and water level time series, the model simulated the entire input wave data (first 12-years) and then repeated the time series until a 20-yr simulation was completed. A nearshore depth of -20 ft NAVD 88 was defined as the input wave water depth.

3. Coastal Structures Data (Existing Conditions Model)

The coastal structures implemented in the existing conditions, post-fill model were the same as those utilized in the 1999-2002 calibration model. Many of the structures on the beach were covered by the beach fill, including the concrete rubble on the beach adjacent to the Little Creek jetty and other locations of rubble. However, these structures were implemented as seawalls, and can be expected to remain nonerodible even after they may become exposed from erosion.

4. Sediment Data (Existing Conditions Model)

Since this modeling analysis was completed after M&N obtained the post-fill sediment analysis report summary, the characteristic post-fill data were used to determine the effective sediment grain size (d50). As stated in **Section II-D** after summarizing the sediment analysis data for East Ocean View (**Appendix E**), it was determined that the sediment grain size characteristics varied between the eastern and western portions of the study area. Therefore, characteristic sediment grain sizes of 0.5 mm for the eastern section of the study area and 0.34 mm for the western portion of the study area were selected for the models based on the existing beach conditions.

5. Boundary Conditions (Existing Conditions Model)

The seaward boundary conditions used in the final existing conditions model were the same as those used in the calibration model, which included a smoothing factor of 35 and an angle offset of +18° for the eastern portion of the study area.

The lateral boundary conditions were the same as those established for the 1999-2002 calibration model, with the exception of the west-left model boundary. This boundary was defined as moving at a rate of -0.012 ft/yr, which was calculated based on the average change in shoreline position between the measured 1995 and 2002 shoreline positions.

6. Model Output (Existing Conditions Model)

Using the established model calibration coefficients including the longshore sediment transport K-factors and the breakwater transmissivity factors, the existing conditions model was run for a 20 year time period. The 20 year simulation time period utilized the nearshore wave data from January 1991- December 2003. The model output was extracted yearly for the month of December to determine the point in time when the initial shifted equilibrium shoreline reached the pre-nourishment shoreline. The pertinent model results which indicated the expected design life will be shown in the following section.

VI. ANALYSIS OF EXPECTED DESIGN LIFE

The expected design life of the beach nourishment project was defined as the time over a 20-year GENESIS model simulation (2003-2023) that the existing, post-fill shoreline (shifted equilibrium position) reached the pre-nourishment, pre-Isabel shoreline position. Through the modeling, it has been determined that the design life will vary across the site and that most areas immediately behind the breakwaters will be somewhat more stable than those areas in-between the breakwaters and where no breakwaters exist. However, the breakwaters were found to be less effective than expected as evidenced by the high transmission coefficient required for the GENESIS model calibration.

The modeling showed that the most critical areas would be near Ships Cabin and immediately west of the breakwater field near the end of the project and near the concrete beach structure located at the middle of the project area just seaward of 25th Bay Street. The models estimate that the shorelines in these areas would reach the pre-project position within 7 – 8 years. Some areas along the study area would have a longer design life, but it is likely that some work would have to be completed in these other critical areas. **Figures VI-1** and **VI-2** show the graphical model results for the 7-year (2010) and 8-year (2011) time periods. **Figure VI-3** shows the general location of these critical areas overlain on the 1999 aerial photography. Yearly model results for the dates up to and including the span of the expected design life (2004-2011) are included in **Appendix J.**

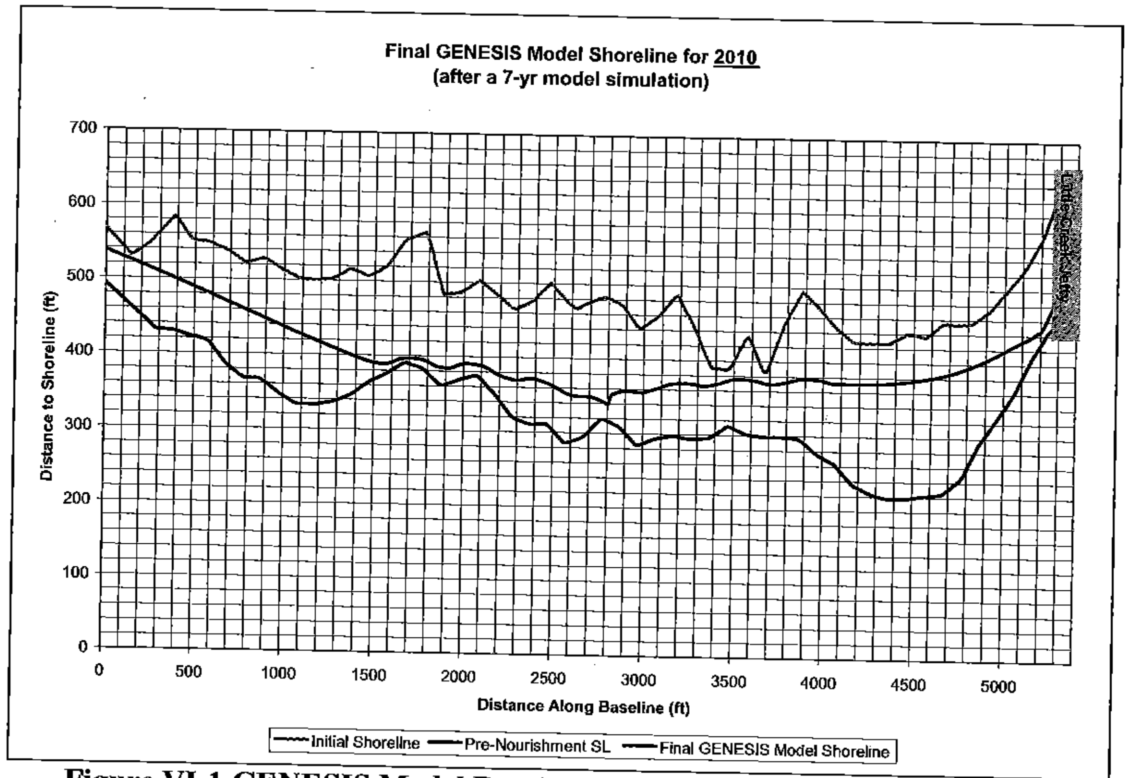


Figure VI-1 GENESIS Model Results for 2010 (7-year model simulation)

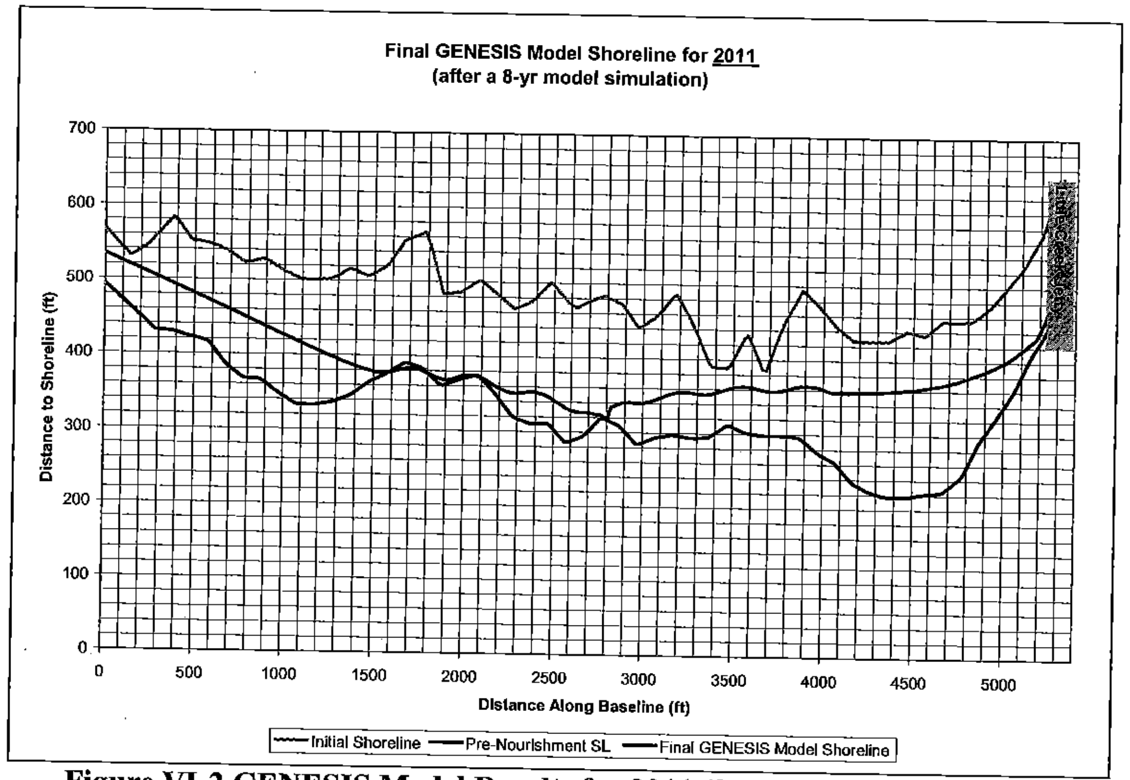


Figure VI-2 GENESIS Model Results for 2011 (8-year model simulation)

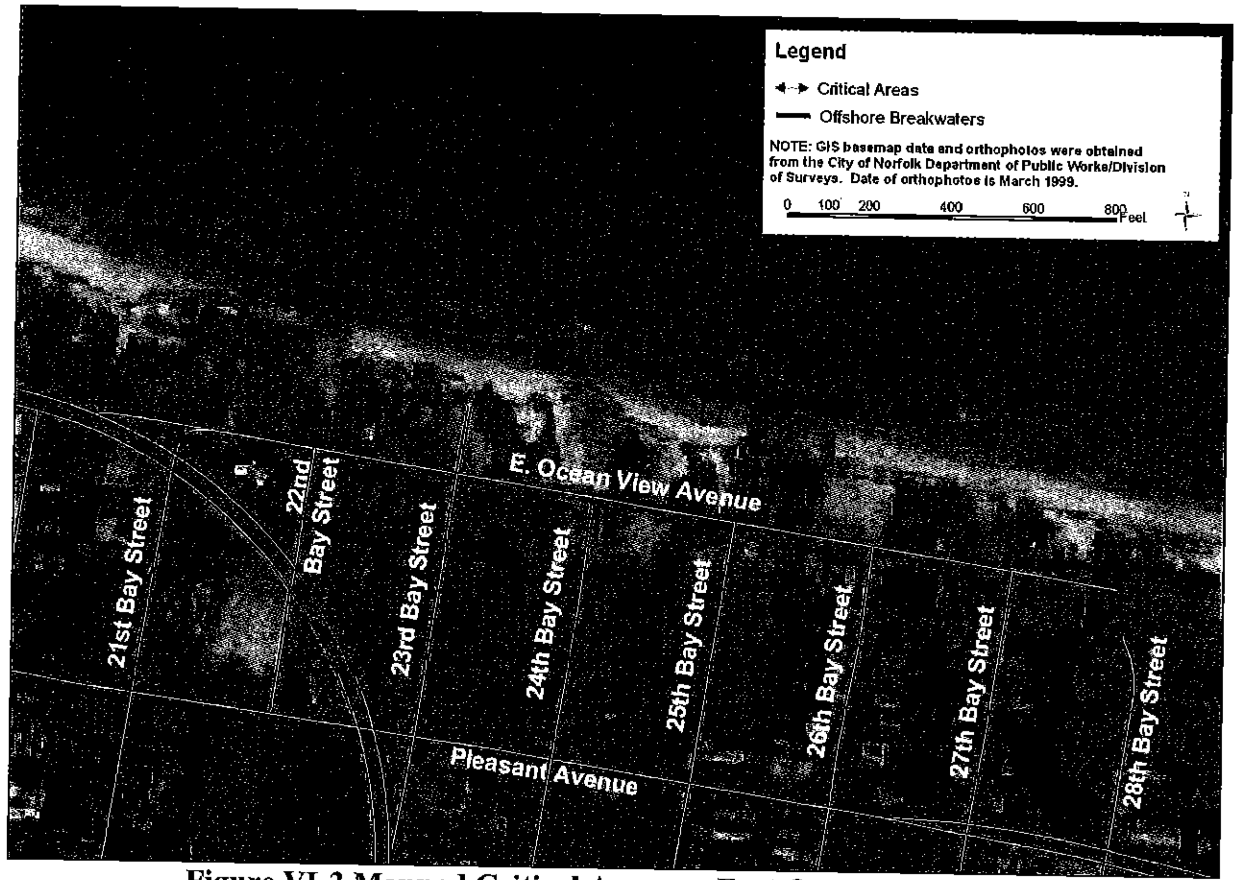


Figure VI-3 Mapped Critical Areas on East Ocean View Beach

Having estimated a project design life of 7-8 years, based on the long-term GENESIS model results, it should also be stated that this design life could be lessened if storm-induced erosion similar to that estimated with the Hurricane Isabel SBEACH model occurs. The Hurricane Isabel SBEACH model output yielded an average erosion setback distance of 35 ft for the representative transects modeled. Therefore, if a major storm event like Isabel occurred in a given year, the expected design life of the nourishment project could be lessened to 4-5 years if the storm event occurred during those years. While recovery of the beach can be expected to occur following storm-induced erosion, if the storm event occurs during year 4-5, the shoreline could erode to the pre-project position at the critical locations. In order to quantify and compare the modeled design life against measured beach changes over time, M&N recommends that the City continue collecting profile surveys every six months. The spacing of the profiles should be such that profiles are collected immediately behind and between the breakwaters (spacing approximately 200 ft).

As stated previously, the GENESIS model wave input time series included representative nearshore sea and swell components for 1991-2003. The expected design life, as defined above, was reached within 8 years of the model simulation. Therefore the waves included in the 8-year time period occurred between January 1991- December 1999 based on the input wave time series. This time span was analyzed to identify the occurrences of storm events which influenced the overall model results and to verify that this time period was typical with respect to the wave climate.

To identify significant storm events that occurred between 1991-1999, the wave data time series was filtered to show only time series measurements which had combined wave heights exceeding 3.5 ft. This height was specified based on a percent exceedance analysis of the nearshore combined wave heights. For this analysis, the combined wave heights were computed as the sum of squares of the sea and swell components for a given time step. The selected dates were compared with a map of Atlantic hurricane paths and a database of storm events listed by the Duck FRF site to identify significant hurricanes and noreasters which were represented in the wave data time series. **Appendix K** includes a table listing all of the storm events which exceeded a 3.5 ft nearshore combined wave height. Some notable storms include the "Halloween Storm" (October 1991), Hurricane Gordon (November 1994), Hurricane Felix (August 1995), Hurricanes Dennis (August 1999) and Hurricane Floyd (September 1999). It should be noted that some storm events, including the "Storm of the Century" (1993) and Hurricane Emily (1993) were not included in the wave data set, because the Duck gage became inoperable during the storm event. However, given the length of the time series and the number of storm events represented, the 8-yr wave conditions simulated were considered a reasonable representation of both long-term average wave conditions and the frequency of storm events. A number of storm events had peak wave heights exceeding or equal to the peak wave height estimated for Hurricane Isabel.

VII. CONCLUSIONS

The objective of this report was to summarize the East Ocean View Beach Nourishment project completed in December 2003 and present the findings of a thorough coastal modeling study to estimate the expected design life of the nourishment project. This report presented the processes of data collection, data transformation through coastal modeling, and short-term and long-term modeling of potential shoreline impacts on the constructed beach.

The overall coastal modeling analysis included the use of SBEACH to evaluate the immediate cross-shore loss of sand over a one year time period and following a storm event represented by Hurricane Isabel. Additionally, GENESIS was used to evaluate the long-term change in shoreline position based upon a twenty-year time period of wave action. Both modeling analyses required complete nearshore wave data time series broken into sea and swell components, which was developed through the transformation of measured wave data at the Duck FRF offshore of North Carolina. The expected design life was estimated by calculating the time period by which the post-project equilibrium shoreline reached the pre-project, pre-Isabel (June 2003) shoreline position. In addition to the analysis of expected design life of the project, separate SBEACH model results gave insight to the impact Hurricane Isabel would have on the nourished project site.

Through the integrated modeling analyses, it was determined that the design life can be expected to vary across the site with two main critical areas: 1) near Ships Cabin and immediately west of the breakwater field near the end of the project and 2) near the concrete beach structure located at the middle of the project area just seaward of 25th Bay Street (see **Figure V-3**). The models estimate that the shorelines in these areas would reach the pre-project position within 7 – 8 years. This design life could be lessened to 4-5 years if storm-induced erosion similar to that estimated with the Hurricane Isabel SBEACH model occurs. This estimate was based on the average erosion setback distance of 35 ft determined through the Hurricane Isabel SBEACH modeling. While recovery of the beach can be expected to occur following storm-induced erosion, if the storm event occurs during years 4-5, the shoreline could erode to the pre-project position at the critical locations. In any case, some areas along the overall study extent can be expected to have a longer design life than defined by the long-term modeling and the potential Hurricane Isabel impacts.

As for future recommendations, M&N strongly suggests that the City consider installing wave gages offshore to acquire more accurate wav data which would allow for greater confidence and efficiency in the decision of future shore protection projects. Also, in order to quantify and compare the modeled design life against measured beach changes over time, M&N recommends that the City continue collecting profile surveys every six months. The spacing of the profiles should be such that profiles are collected immediately behind and between the breakwaters (spacing approximately 200 ft).

In summary, the East Ocean View beach nourishment project was implemented successfully, providing crucial shoreline stabilization following the severe storm impacts which threatened the shoreline and structures impending on it. This project can be expected to have a reasonable design life, which will provide for further storm protection and mitigate potential damages posed by future long-term erosion and short-term storm induced erosion. The modeling analyses performed in the comprehensive study of East Ocean View Beach not only provided more insight to the expected design life of the beach nourishment project, but improved the overall understanding of the coastal processes occurring in and around the study area, which will aid in future decision making related to shoreline stabilization and improvement. Furthermore, the data collection and transformation involved in this study will be utilized in subsequent studies of other sections of the City of Norfolk shoreline.